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PHASE ONE - SEWERAGE PLANNING STUDY

CSA NO. 9 - LOS OSOS, BAYWOOD PARK, CUESTA-BY-THE-SEA

PREPARED FOR:

COUNTY ENGINEERING DEPARTMENT
SAN LUIS OBISPO COUNTY, CA.

MAY 1986

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Draft

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May 1986

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TABLE OF CONTENTS

APPENDIX A REFERENCES

APPENDIX B PROJECT STAFF

APPENDIX C SOUTH BAY LAND USE PLAN EXCERPTS

APPENDIX D RWQCB LETTER RE PROBABLE WASTE DISCHARGE REQUIREMENTS

APPENDIX E TITLE 22 WASTEWATER RECLAMATION CRITERIA

APPENDIX F PRESSURE SEWER INSTALLATIONS

APPENDIX G VARIABLE-GRADE GRAVITY SEWER INSTALLATIONS

APPENDIX H MORRO GROUP LETTER REPORT RE DISPOSAL SITE 6

APPENDIX I SLO COUNTY MEMORANDUM RE BLACK LAKE SEPTIC TANK EFFLUENT

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

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The Regional Water Quality Control Board, Central Coast Region (Regional Board) has concluded that the quality of groundwater in the Los Osos/Baywood Park area of San Luis Obispo County is being degraded by excessive nitrate and bacteria concentrations. The primary source of these contaminants has been identified as septic tank effluent discharged from individual septic tank/leach field systems which currently provide wastewater treatment and disposal throughout the area. In order to mitigate the decline in quality of groundwater in the area, the Regional Board adopted an amendment to the Water Quality Control Plan, Central Coast Basin (Basin Plan) which prohibits individual and cluster septic tank/leach field systems in the Los Osos/Baywood Park area as of 1 November 1988. The Basin Plan amendment also established a time schedule for design and construction of a wastewater collection system and centralized treatment and disposal system for the area to replace the septic tank/leach field systems (Reference 1.1).

The purpose of this Phase I Planning Study is to define the most cost-effective and technically feasible methods for collection, treatment, and disposal of sewage from the Los Oso/Baywood Park area in accordance with the requirements of the amended Basin Plan and the Regional Board. The results of this study will be used as the basis for subsequent preliminary design and financial planning for the sewerage project.

HISTORY

The history leading to this planning study for the Los Osos/Baywood Park sewerage project is long and eventful. A synopsis of the history is as follows:

- June 1971. Interim Basin Plan prepared by the Regional Board prohibits on-site septic tank waste discharges in the Los Osos/Baywood Park area after 1 July 1974.

- June 1974. San Luis Obispo County (County) protests 1 January 1980 Los Osos/Baywood Park discharge prohibition contained in the Draft Basin Plan. The County contends that feasibility studies are necessary and initiates a groundwater monitoring program.

- April 1975. The Basin Plan requires the establishment of a septic tank maintenance district for the area. The Basin Plan also requires that studies be undertaken to determine the necessity of sewering the area and to determine the characteristics of the groundwater basin.

- October 1978. The Regional Board requests "emergency assistance" from the State Water Resources Control Board (SWRCB) to evaluate how on-site sewage system discharges affect water quality within the Los Osos/Baywood Park groundwater basin.

- October 1979. The SWRCB study, prepared by Mr. Richard Zipp, is completed. The study concludes that shallow groundwater is being degraded by effluent from sewage leach fields and that continued development would compound the existing problem. The study also concludes that little mixing of septic tank effluent with the deep groundwater (which constitutes the primary water source for the area) had occurred at that time but that increased pumping from the deeper groundwater to accommodate growth could cause mixing resulting in degradation of this water (Reference 1.3).

- November 1979. The County and Regional Board agree that the scope of the Richard Zipp study was inadequate and that a Phase

I study would be performed to identify any water quality problem and its source.

- November 1980. County authorizes Brown and Caldwell to proceed with the Phase I study.

- April 1983. Brown and Caldwell completes the Phase I study which concludes that shallow groundwater in the area is contaminated with nitrate, that wastewater discharge from individual septic tank/leach field systems is the primary cause of the high nitrate concentrations and that as the population of the area increases, wastewater discharges and water demand will increase, thereby accelerating degradation of groundwater quality (Reference 1.4).

- June 1983. The Regional Board authorizes the County to proceed with the Phase II Facilities Plan Study Project Report and Environmental Impact Report for the sewerage project. The County authorizes Brown and Caldwell to proceed with preparation of these reports.

- September 1983. The Regional Board adopts Resolution 83-13 amending the Basin Plan to prohibit individual or cluster septic tank/leach field systems in the area and to set a schedule for design and construction of a sewerage project for the area.

- January 1984. Brown and Caldwell completes the Phase II study which recommends an area-wide conventional gravity sewer collection system, wastewater treatment at a single community treatment plant utilizing an extended aeration oxidation ditch process with partial nitrogen removal (45 to 25 mg/L as NO_3), and land disposal through percolation ponds to the shallow groundwater. The Phase II study estimated cost for the project was $28.6 million, of which $3.8 million was expected to be funded by EPA and SWRCB grants. The monthly service charge to a typical user was projected to be $60 per month (Reference 1.5).

- March 1984. A public hearing is held to discuss the Phase II report.

- September 1984. Brown and Caldwell issues a supplement to the Phase II report to address public and Regional Board comments and to revise the project EIR.

- February 1985. The Morro Group is retained by the County to prepare an EIR for the project.

- April 1985. Engineering-Science is retained by the County to perform a planning study (Phase I) and preliminary design (Phase II) for the sewerage project in coordination with the Morro Group.

The project history has included debates about the hydrogeologic characterististics of the groundwater basin, the merits of preserving the quality of the shallow groundwater which currently is not used as a primary water source, the potential for degradation of the deeper groundwater which is the primary water source for the area, and the use of other types of systems in lieu of a sewerage system to protect groundwater quality in the area. An ongoing concern of the County and of Los Osos/Baywood Park residents is the financial implication of an area-wide sewerage project. This financial implication was emphasized by the Phase II Facilities Plan user fee estimate of $60 per month. However, in light of the Regional Boards' 1983 amendment to the Basin Plan, a sewerage project appears inevitable. This Phase I Planning Study has therefore been undertaken to determine the most cost-effective sewerage system to serve the area.

AUTHORIZATION

On 12 March 1985, the County Board of Supervisors authorized the County Engineer to proceed with negotiations with Engineering-Science (ES) for an agreement to perform a planning report and a preliminary engineering report for the Los Osos/Baywood Park sewerage project. On 23 April 1985, the County Engineer recommended that the Board of Supervisors enter into the agreement which had been negotiated by the County and ES. On April 1985, the Board of Supervisors adopted a resolution approving the agreement and authorized and directed the chairman of the Board to execute the agreement. A draft report was submitted to the County for review in July 1985. Completion of the report was delayed until April 1986 pending geotechnical investigation of wastewater disposal sites.

PROJECT OBJECTIVES

The objective of the Los Osos/Baywood Park Sewerage Project is to provide a sewage collection, treatment, and disposal system which will perform reliably and which will impose the minimum level of financial burden to users of the system. The purpose of the project is to protect and preserve the quality of groundwater in the Los Osos/Baywood Park area, as mandated by the Regional Board under the authority of the Porter-Cologne Act. In addition, a paramount objective for the project is to maximize recharge of the Los Osos groundwater basin with treated effluent from the project wastewater treatment facilities.

SCOPE OF WORK

The agreement between the County and ES provides that ES perform two separate phases of work: Phase I Planning Study and Report and Phase II Preliminary Engineering and Basis of Design Report. This report embodies the work performed as part of the Phase I Planning Study.

The purpose of Phase I is to identify the most cost-effective and technically feasible method of collecting and treating sewage and disposing of effluent and sludge in a manner acceptable to the Regional Board. Concepts to be studied and evaluated include the the following:

Sewage Collection System

- Conventional gravity sewers
- Variable - grade gravity sewers
- Pressure sewers
- Combinations of the above

Treatment Plant for Denitrification

- Oxidation ditch
- Batch reactor
- Physical-Chemical

Effluent Disposal

- Multiple sites for most probable impact on recharge of the aquifers and control of sea water intrusion.

In recognition of the interest of members of the Los Osos/Baywood Park community in the selection of alternative sewerage systems for the area, every effort will be made to evaluate other alternatives raised by community representatives provided that such alternatives are reasonable in number and complexity with respect to available data.

Specifically excluded from the Phase I work are the following:

- Soils investigation
- Surface and subsurface hydrological investigations
- Surveying
- Water quality analysis
- Environmental impact (analysis) assessment

The Morro Group, an environmental consulting firm, has been hired by the County to prepare the Environmental Impact Report (EIR) for the proposed project. The work of the Morro Group will be underway concurrently with the Phase I work, and information developed by The Morro Group and ES is to be shared and conjunctively used by them to complement the preparation of the Phase I Report and the preparation of the EIR.

Significant cost estimates will be prepared by ES for comparison of alternatives in order to select the more cost-effective and technically feasible alternatives based upon both capital costs and operation and maintenance costs. These cost estimates are not intended to be adequate for detailed financial planning. ES will attend a public hearing planned at the end of Phase I as part of this study effort, and will report on the results of the hearing by means of addenda rather than by editing and reproducing the original report. Based upon the results of

Phase I, sufficient preliminary engineering will be performed in Phase II to permit preparation of detailed cost estimates to be used by the County in finalizing a financing program.

RELATED REPORTS

The data base for Phase I was the "Phase II Facilities Planning Study," dated January 1984, and the "Supplement to the Phase II Facilities Planning Study Project Report and Environmental Impact Statement," dated September 1984, prepared by Brown and Caldwell for the County. Appendix A contains a listing of all other references for this Phase I Report.

ACKNOWLEDGMENTS

Engineering-Science is appreciative of the valuable assistance and guidance provided by the County during the conduct of the study. In particular, ES would like to acknowledge the efforts of Mr. George Protopapas, County Engineer; Mr. Clint Milne, Deputy County Engineer; and Mr. George Gibson, project manager for the County on this project.

PROJECT STAFF

This Phase I Planning Study has been prepared under the direction of Mr. T. G. Cole. Project engineers and their contribution to the study are listed in Appendix B.

CHAPTER 2

STUDY AREA CHARACTERISTICS

CHAPTER 2

STUDY AREA CHARACTERISTICS

STUDY AREA BOUNDARIES

The overall study area for the project is located on the central coast of California in San Luis Obispo County and includes the westerly draining half of Los Osos Valley and Clark Valley. The study area is bounded on the west by Morro Bay and El Estero Bay, on the north by a line of hills known as Park Ridge, on the south by the Irish Hills, and on the east by the easterly-draining half of Los Osos Valley. Figure 2.1 shows the location and boundaries of the study area.

The service area for the proposed sewerage project is County Service Area No. 9 (CSA No. 9). The service area is located on the southern tip of Morro Bay at the mouth of Los Osos Valley, approximately 12 miles northwest of the City of San Luis Obispo and a few miles south of the City of Morro Bay. The CSA No. 9 area is also referred to as Los Osos, Los Osos/Baywood Park, and South Bay, and in addition to Los Osos and Baywood Park includes the neighborhood communities of Cuesta-by-the-Sea, Bayview Heights, Creekside, Upland, Sunset, and Highland. The Regional Water Quality Control Board (RWQCB) zone of septic tank prohibition is within CSA No. 9 and encompasses almost all of the developed area of CSA No. 9. CSA No. 9 boundaries and the prohibition boundaries are shown in Figure 2.2.

TOPOGRAPHICAL AND GEOGRAPHICAL SETTINGS

The Los Osos Valley is a relatively flat alluvial plain lying between the two roughly parallel ranges of low hills mentioned above.

FIGURE 2.1

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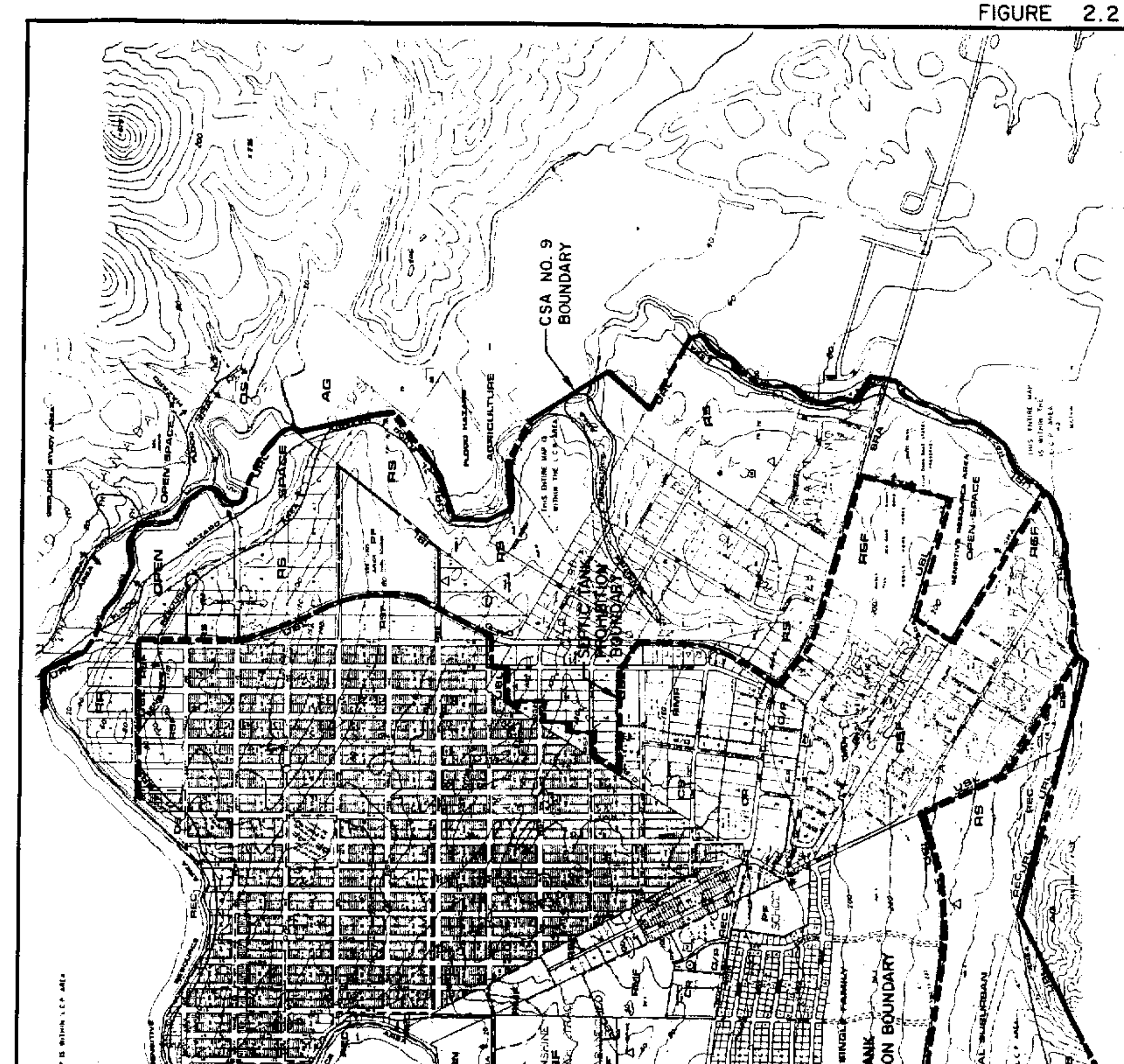

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The Irish Hills to the south vary in elevation from near sea level at the coast to 1,500 feet at the southeast corner of the study area. The Irish Hills drop gradually from 1,500 feet to a peak of 1,300 feet near the middle of the southern boundary of the study area. At that point they decline in height more rapidly to near sea level at a point just below the southern tip of Morro Bay estuary. Park Ridge on the northern boundary of the study area is composed of a linear series of volcanic necks and ranges in elevation from 800 feet to 900 feet, and drops abruptly at the west end from Cerro Cabrillo Peak to the Morro Bay mud flats. Clark Valley is a smaller and higher valley than Los Osos Valley. It is located south of Los Osos Valley in the Irish Hills, isolated by a ridge which runs along approximately two-thirds of the southern boundary of the study area (Reference 2.1.).

The topographical characteristics of the service area (CSA No. 9) are highly variable due to the local convergence of the Irish Hills, Los Osos Valley, the Pacific Ocean, Morro Bay, and Los Osos Creek. The southern portion of the service area consists of the trailing end of the Irish Hills. This area slopes continuously downward from an elevation of approximately 800 feet to an elevation of 20 feet at an average slope of about 6 percent.

The southwestern portion of the service area includes the drainage basins of two tributaries of Los Osos Creek. The topography of this area slopes downward to the tributaries from an elevation of approximately 200 feet. The area between the two tributaries is relatively flat around the 200-foot elevation or is generally sloping toward the ocean and Morro Bay.

The topography of the central and northern portions of the service area consists of rolling, undulating terrain representative of windblown sand dune formations. Much of the central area has no well-defined drainage pattern. During wet weather, many isolated depressions in this central area experience ponding (Reference 2.1). There are several northwest-to-southeast aligned ridges in the north half of the sand dune area which also tend to disrupt drainage patterns. Wet weather ponding occurs in the low-lying areas between ridges.

Along the shoreline of much of the service area, the topography is flat and low. Along the northwest boundary of the service area, however, the terrain drops steeply to the shoreline. The northeastern portion of the service area is tributary to Los Osos Creek, dropping relatively steeply from higher sand dune formations. The topography of the study area and the service area can be seen on Figures 2.1 and 2.2, respectively.

SERVICE AREA LAND USE AND POPULATION

During the 1970s, the service area experienced a period of high population growth. Most of the area had already been subdivided into residential lots but remained undeveloped into the early 1970s. Growth in the population of the area was able to occur at a rapid pace in the 1970s as the subdivided lots were developed. From 1970 to 1980, the population of the service area increased 214 percent from 3,500 to 10,900. The current population is estimated to be 13,100 which represents a 3.7 percent average annual growth rate since 1980. The County estimates that the ultimate population capacity of the service area is 28,200 (Reference 2.1). This capacity includes much of the unsubdivided land in the service area which remains designated for future residential development. Table 2.1 lists the current number of residential units within the service area.

TABLE 2.1

CSA NO. 9 DWELLING UNITS AS OF JUNE 1985

Type of Unit	Number of Units
Single family dwelling	3,711
Multiple family dwelling	1,334
Suburban residential	241
Rural residental	8
Total	5,294

Reference 2.2.

Land use in the service area is primarily residential with some neighborhood-serving commercial development. There is very little industrial development in the service area. Densities of residential development range from very high density urban to low density rural. Much of the area was subdivided in the late 1880s into small size lots (25 feet by 125 feet), which are substandard by current County standards. There are a total of approximately 9,000 lots in the service area, of which 6,000 are of the substandard size. The County currently has a lot consolidation policy which discourages development on substandard size lots in the area. To account for this prohibition, and for the fact that often a single house was built on two substandard lots, it was assumed for this study that from the 6,000 substandard lots, only 4,500 residential units will be built. This 25 percent reduction is probably conservative because most of the development in the Baywood Park subdivision of substandard lots has occurred on multiple lots. A field survey is recommended to verify the actual number of existing dwelling units which have been constructed on 25-foot wide lots. The number of users in the sewage collection system will significantly impact the overall sewerage system cost, as discussed in Chapter 6.

Appendix C contains excerpts from the County Land Use Plan which relate to CSA No. 9 (South Bay), including a description of land use categories, existing development, and future development plans.

Population projections for use in determining design sewage flows discussed in Chapter 3 were developed considering the historical and ongoing growth patterns for the service area as noted above. The current population of the study area is now approaching 50 percent of the projected saturation population. The population at the beginning of the 1970's high growth period was only about 10 percent of the saturation population. As the population of the area increases, the rate of additional development will decrease and the population will assymptotically approach the saturation level.

A population projection was developed for the service area. The population projection for the year 2000 is 18,700, or about two-thirds of the saturation population of 28,200. The year 2000 population projection was therefore selected as the Stage I design population for those facilities for which construction phasing is appropriate (such as treatment and pumping facilities). Stage II construction will require a 50-percent expansion of the sewerage facilities in the year 2000 which will be a convenient expansion increment and which will enable the facilities to accommodate the ultimate population of the service area. The 18,700 population in the year 2000 represents an average annual growth rate of 2.4 percent from current population levels.

WATER RESOURCES

Basin Characteristics

The Los Osos hydrologic basin comprises a sub-area of the San Luis Obispo hydrologic sub-unit of the Central Coastal hydrologic study area. Total areal extent of the Los Osos hydrologic basin is approximately 18,000 acres, and includes the westerly draining half of the Los Osos Valley and part of Clark Valley (Reference 2.4). The basin is bounded on the north by a line of hills known as Park Ridge, and on the south by the Irish Hills. The basin extends westerly beneath Morro Bay and the ocean for an unknown distance, and easterly from the Morro Bay estuary inland approximately 6.5 miles. The San Luis Obispo County Engineering Department suggests that some subsurface groundwater flow originates from outside the basin boundaries and that the shallow groundwater aquifers within the basin are unconfined while the deeper formations are semi-confined or confined (Reference 2.5).

Surface Water Hydrology

The surface water features of the Los Osos hydrologic basin include Los Osos Creek and its tributaries, Eto Creek, a few small lakes and impoundments, and several less prominent depressions in the western sand dunes. During the average year, approximately 6,610 acre-feet of surface runoff is generated, as estimated by the Department of Water Resources (DWR) (Reference 2.11). Of this amount, the DWR contends that approximately 2,500 acre-feet flow into Morro Bay and the remaining 4,110 acre-feet percolate into the ground.

Subsurface Hydrology

Groundwater occurs principally in Holocene to Pliocene sediments of the basin. These sediments make up a fairly thick sequence of fresh water bearing sediment that have been categorized into two groundwater zones (References 2.1, 2.3, 2.4). The upper aquifer zone includes 100-200 feet of the old dune sand deposits and the upper portion of the Paso Robles Formation. The lower aquifer zone is thought to include the remaining portion of the Paso Robles Formation. These zones are thought to be separated by a confining layer of low permeability material (References 2.1, 2.3, 2.4, 2.7). The upper aquifer zone contains a shallow unconfined water table aquifer, while the lower zone contains groundwater under semi-confined conditions.

Recharge for the upper aquifer zone is principally from direct percolation of rainfall and septic tank discharge. The groundwater levels in the upper aquifer sediments generally follow the surface topography. Discharge from the upper aquifer zone is to Los Osos Creek, Eto Creek, and Morro Bay.

Below the upper aquifer zone is a transition zone made primarily of sediment of the Paso Robles Formation. The transition zone may be comprised of a fairly continuous clay layer which appears in the upper horizons of the Paso Robles Formation (References 2.3, 4, and 7). This series of sediments is thought to be eroded in the vicinity of the Los Osos floodplain, and the alluvium in the floodplain is thus thought to be in hydraulic communication with the Paso Robles sediments (Reference 2.7). The transition zone extends to a depth of approximately 50 feet below sea level at the western edge of the basin, and rises in elevation to the east where it apparently pinches out in the vicinity of Los Osos Creek. The transition zone may provide an effective aquitard, restricting flow between the upper Old Dune Sand aquifer zone and the lower (Paso Robles Formation) zone.

The Paso Robles Formation includes a sequence of alternating layers of sand, gravel, and clay that forms a wedge-shaped series of confined to semi-confined aquifers extending from west of Morro Bay to just east of the Los Osos Creek floodplain where it outcrops. The thickness of the Paso Robles Formation sediments at the eastern edge of Morro Bay is approximately 325 feet, but they are believed to thin to 100 feet or less at the eastern edge of the groundwater basin (Reference 2.3).

Recharge to the lower aquifer zone is believed to be primarily along and east of Los Osos Creek (References 2.3 and 7). Additional recharge occurs through Warden Lake and the unnamed tributary which extends from Warden Lake to Los Osos Creek. Discharge from the lower aquifer zone occurs along a front extending more or less north-south across Morro Bay.

Hydrologic Budget

A generally accepted procedure in the analysis of groundwater basins is the development of a hydrologic budget that balances inflow and outflow, taking into account natural recharge, return waters from urban and agricultural uses, and outflow from the basin. Hydrologic budgets were prepared for 1972 and 1980 conditions by Brown and Caldwell in 1983 (Reference 2.3). A re-evaluation of the Brown and Caldwell conclusions considering additional information as it is collected in this area is currently being conducted by the Morro Group.

Realizing that these hydrologic budgets have treated the groundwater basin as a single aquifer unit and not as a two-aquifer system, as revealed in more recent investigations (References 2.3, 4, and 7), the Morro Group is preparing an evaluation of a preliminary hydrologic budget for the basin based on a two-aquifer system. It should be emphasized that this budget should be considered an estimate until the preparatory assumptions are further substantiated.

Groundwater Quality

The quality of waters in the upper and lower aquifer zones of the Los Osos hydrologic basin has been studied in detail by Brown and Caldwell (1983) in their Phase I Water Quality Management Study. The main conclusion of their investigation was that, while the lower aquifer maintains good water quality, the upper part of the upper aquifer zone contains concentrations of nitrate that in some areas exceed the State of California standard for domestic use of 45 mg/l.

WASTEWATER MANAGEMENT

Wastewater disposal in the project service area is presently accomplished with individual, on-site disposal systems consisting of septic tanks and leach fields. Community or cluster septic tank/leach field systems are also employed in the service area to provide wastewater disposal to a group of users. Sewage from users in the group is collected by gravity sewers and transported to the community septic tank/-leach field system.

Each of the four mobile home parks in the service area (Sea Oaks, Sunny Oaks, Daisy Hill, amd Morro Shores) has a community septic tank/ leach field system (Reference 2.8). The total number of units in the mobile home parks is 495 (Reference 2.9). The Bayridge Estates subdivision utilizes two community septic tank/leach field systems, each of which serves half of the 234 residential lots in the subdivision (Reference 2.8). The Vista de Oro subdivision also has a community sewerage system to serve its 68 residential lots (Reference 2.8).

Owing to seasonal occurrence of high groundwater in several places within the service area and to high density of development, several septic tank failures (probably above normal) have been reported in the service area (Reference 2.10).

POLITICAL INSTITUTIONS

CSA No. 9 was formed in 1973 to consolidate a number of single purpose districts. As a County Service Area, CSA No. 9 is governed by the San Luis Obispo County Board of Supervisors. The westerly coastal portion of the study area, including the project service area, lies within County Supervisorial District No. 2. The southerly fringe of the study area lies within Supervisorial District No. 3.

Services authorized for CSA No. 9 include police protection, structural fire protection, local parks, recreation or parkway facilities and services, tree maintenance within public right-of-way streets, lighting and sweeping, storm drains and drainage, sewage disposal and treatment, water, ambulance, irrigation, and solid waste disposal. Services provided are financed through zones of benefit upon request of the property owners. Presently, services provided by CSA No. 9 include water, fire protection, drainage maintenance, street lighting septic tank maintenance and park maintenance. Services are as follows:

Area	Services
CSA 9A	Water
CSA 9B	Fire Protection
CSA 9D	Drainage, street lights, Cabrillo Estates only
CSA 9E	Street lights, sewage treatment, drainage, Vista de Oro only
CSA 9F	Street lights, sewage treatment, drainage, Bayridge Estates only
CSA 9G	Drainage maintenance zones in three locations

In order to provide public input to the County regarding decisions and policies made involving CSA No. 9, two community advisory groups have been formed. County Service Area No. 9 Advisory Group is concerned with activities involving CSA No. 9 in general. The South Bay Water Quality Advisory Group (SBWQAG) was formed specifically to satisfy federal public participation requirements for Clean Water Grant funding of facilities constructed to mitigate water quality problems in the area. SBWQAG members were appointed by the County Board of Supervisors to review studies made concerning water quality in the area to provide information to the public, to receive public comment and to advise the Board of Supervisors accordingly.

The project service area is entirely within the Coastal Zone. A sewerage project in the area would therefore be subject to a coastal permit from the California Coastal Commission.

CHAPTER 3

WATER SUPPLY AND WASTEWATER CHARACTERISTICS

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WATER SUPPLY

The source of water for the study area is groundwater which exists in aquifers below Los Osos Valley. Local surface water resources are not utilized and no water is currently imported to the area. Groundwater is replenished by direct infiltration of precipitation, streambed infiltration and recharge of septic tank effluent in various portions of the study area. The aquifers also receive recharge from leached irrigation water from agricultural activities in the eastern portion of the Study Area.

There are three municipal water purveyors which have distributed chlorinated well water to the Los Osos/Baywood Park area since the early 1950s: County Service Area 9A (1951); Cal Cities (Southern California Water Company), a private water company (1954); and S&T Water Company, a mutual water company (1955). Rural domestic and agricultural water supply is provided by private wells. The nine-hole golf course located in the southwestern portion of the service area has private wells for golf course irrigation and to maintain a small impoundment. Some older, shallow domestic wells in areas now served by water purveyors are maintained for landscape irrigation.

Additional water can be made available from the State aqueduct when the coastal branch of the aqueduct is completed (Reference 3.1), or from another project as identified in the County's Master Water Plan.

WATER DEMAND

A review of existing documentation indicates that average domestic water demand in the service area is currently about 0.167 acre-feet per capita per year, or 148 gallons per capita per day (gpcd). The per capita water demand was calculated by dividing the water production of the three municipal water purveyors by the estimated population of the service area. Water used for golf course irrigation was specifically excluded from the per capita water demand calculation. Per capita water use in the area appears to have stabilized due to water conservation measures implemented during previous drought periods including the use of water saving plumbing fixtures for new construction (Reference 3.2). The golf course within the project service area historically consumes 110 acre-feet per year for irrigation (Reference 3.1).

Projected future urban water demand in CSA No. 9 is presented in Table 3.1. The basis for the population projections used to develop the water demand projections in Table 3.1 is described in Chapter 2.

TABLE 3.1

PROJECTED URBAN WATER DEMAND

	Year 2000	Ultimate Growth
Population	18,000	28,200
Residential water demand (AF/yr)	3,010	4,710
Golf course water demand (AF/yr)	110	110
Total urban water demand (AF/yr)	3,120	4,820

PROJECTED WASTEWATER FLOWS

A review of existing documentation indicates that a per capita flow rate of 85 gpcd should be assumed for this project. Two approaches were used to calculate the per capita sewage flow rate. The first approach utilized the Inventory of Waste Dischargers for the service area which was prepared by the County by applying typical unit flow rates to all sewage producers in the area (Reference 3.1). The total sewage flow from this inventory was divided by the population used in the inventory resulting in a sewage flow rate of 85.1 gpcd. The second approach utilized the per capita water use developed previously in this chapter and the assumption that 58 percent of the water use becomes wastewater (Reference 3.3). The 58 percent non-consumptive factor compares favorably with the nearby cities of San Luis Obispo and Arroyo Grande at 60 percent and 56 percent, respectively. Based on a water demand rate of 148 gpcd and 58 percent non-consumptive factor, the sewage flow rate is 85.8 gpcd. This rate appears to be appropriate given the social and economic characteristics of the community.

It should be noted that no field data have been collected to verify the 85 gpcd rate. Collection of field data and development of a more accurate flow projection is not considered necessary for selection of the most cost-effective project. Wastewater flows in the year 2000 (Stage I) and at ultimate development (Stage II) of the service area (CSA No. 9) are projected as shown in Table 3.2. The basis for the population projections used is described in Chapter 2.

TABLE 3.2

PROJECTED WASTEWATER FLOWS

	Stage I	Stage II
Year	2,000	ultimate
Population	18,000	28,200
Per Capita Flow (gpcd)	85	85
Average Dry Weather Flow, ADWF (mgd)	1.6	2.4
Peak Dry Weather Flow, PDWF, (mgd)	3.2	4.8
Wet Weather Infiltration + Inflow, I/I (mgd)		
Gravity Sewers	0.7	0.9
Combination Sewers[a]	0.4	0.5
Pressure Sewers	0.0	0.0
Peak Wet Weather Flow, PWWF (mgd)		
Gravity Sewers	3.9	5.7
Combination Sewers[a]	3.6	5.3
Pressure Sewers	3.2	4.8

[a]Assumes 60% gravity sewers and 40% pressure sewers.

The average dry weather flow (ADWF) of 1.60 MGD is used in process sizing of the treatment alternatives considered in this report. The peak dry weather flow (PDWF) is projected based on a 2.0 peaking factor, which is typical for communities of this size. Infiltration and inflow (I/I) is projected for gravity, pressure, and combination (60% gravity and 40% pressure) collection systems as discussed in Chapter 6. Use of pressure sewers greatly reduces I/I. The peak wet weather flow (PWWF) is considered in the hydraulic sizing of the treatment facilities to insure that high flows during rainfall events can be treated at the plant without violating effluent quality requirements.

PROJECTED WASTEWATER QUALITY

Three types of wastewater (or a combination of them) could potentially be collected and/or treated by project facilities: raw wastewater, septic tank effluent and septage. The characteristics projected for each of these are presented in Table 3.3, as well as the characteristics of the treatment plant influent resulting from the gravity, pressure, and combination collection systems developed in Chapter 6. The treatment plant influent characteristics for the combination collection system have been used in sizing the treatment facilities described in Chapter 7. It is assumed that the wastewater is primarily from residential sources, with some commercial discharge and no industrial discharge.

TABLE 3.3

CHARACTERISTICS OF TREATMENT PLANT INFLUENT

Constituent	Raw Wastewater (mg/L)	Reference	Septic Tank Effluent (mg/L)	Reference	Septage (mg/L)	Reference	Treatment Plant Influent[a] G	C	P
5-Day Biochemical Oxygen Demand (BOD_5)	220	3.3,4	130	3.6,7	10,000	3.5,8,9 10,11	220	215	206
Total Suspended Solids (TSS)	220	3.3,4	80	3.6,7,8	15,000	3.5,8,9 10,11	220	210	196
Ammonia Nitrogen (NH_3-N)	40	3.3,5	40	3.7,8	150	3.5,8 9,11	40	40	40

[a] G assumes 100% raw wastewater.
C assumes 60% raw wastewater + 40% septic tank effluent and corresponding septage.
P assumes 100% septic tank effluent and corresponding septage.

254/5

CHAPTER 4

WASTEWATER DISCHARGE AND TREATMENT REQUIREMENTS

CHAPTER 4

WASTEWATER DISCHARGE AND TREATMENT REQUIREMENTS

WASTEWATER DISPOSAL/REUSE METHODS

Wastewater discharge and treatment requirements are determined based upon the ultimate destination of the final effluent. Potential wastewater disposal and reclamation/reuse proposals for this project include the following:

- Alternative 1 - Dry season discharge to Los Osos Creek with percolation to the groundwater basin (lower aquifer),
- Alternative 2 - Wet weather discharge to Los Osos Creek,
- Alternative 3 - Percolation ponds with groundwater recharge (upper aquifer),
- Alternative 4 - Landscape irrigation (golf course, cemetery, park, etc.), and
- Alternative 5 - Agricultural irrigation.

Alternatives 1 and 3 are the stated goals of the County, if feasible, because of their potential for groundwater recharge to the Los Osos Basin (Refer to Chapter 5). Alternatives 4 and 5 would replace current use of pumped groundwater, and thus would result in a reduction of groundwater use.

REGULATORY AUTHORITIES

Regional Water Quality Control Board (RWQCB)

The RWQCB sets discharge requirements for all facilities which discharge treated wastewater. The Central Coast RWQCB has issued probable waste discharge requirements for the Los Osos facilities (full text is presented in Appendix D). A summary of RWQCB probable requirements for discharge to Los Osos Creek and to percolation ponds is presented in Table 4.1.

TABLE 4.1

RWQCB PROBABLE DISCHARGE REQUIREMENTS

Parameter	Los Osos Creek	Percolation Ponds
BOD_5, mean (mg/l)	10	60
TSS, mean (mg/l)	10	60
Turbidity, mean (NTU)	2	--
Total nitrogen (mg/l as N)	5	5
Coliform bacteria, 7-day median (MPN/100 mL)	2.2	2.2[a]
Dissolved oxygen (mg/l)	5	--
Chlorine residual (mg/l)	undetectable	--

[a]Measured in receiving groundwater.

The key RWQCB probable discharge standards are those for nitrogen (including nitrate) and coliform bacteria. Most treatment processes producing the required nitrate and coliform levels will also produce the low BOD_5 and TSS concentrations required for discharge to Los Osos Creek. Therefore, the 60-60 mg/L BOD_5-TSS requirements for discharge to percolation ponds are not critical to the treatment plant design. For purposes of design, the 10-10 mg/L BOD_5-TSS requirements will be used for the treatment plant discharge standard regardless of disposal method. The total nitrogen standard of 5 mg/l (as N) is tied to a groundwater quality objective of 5 mg/l (as N) nitrate. Factors affecting groundwater nitrate concentration include quantity of natural and irrigation-related groundwater recharge, nitrate concentration in natural and irrigation-related recharge, and nitrate removal during percolation through soil. Groundwater monitoring during project operation may indicate that groundwater quality objectives can be met with a treatment plant effluent nitrate concentration above 5 mg/l (as N).

California Department of Health Services (DOHS)

The RWQCB will incorporate the requirements and recommendations of the DOHS into the discharge requirements for the treatment facilities. DOHS wastewater reclamation criteria are detailed in the California Administrative Code, Title 22, Division 4 (full text presented in Appendix E). DOHS requirements for reclaimed water used for groundwater recharge (Alternatives 1 and 3), landscape irrigation (Alternative 4), and agricultural irrigation of food crops (Alternative 5) are presented in Table 4.2.

TOXIC ORGANIC COMPOUNDS

In the event that toxic organic compounds are found in wastewater from Los Osos in quantities unacceptable to the RWQCB, then an activated carbon treatment process to remove the compounds will be required, either in lieu of or after a particulate removing filter. However, it is assumed that DOHS and RWQCB requirements for removal of toxic organic compounds are not applicable to Los Osos. These compounds are generally associated with industrial discharges to a treatment system. As discussed in Chapter 2 and 3, Los Osos is primarily a residential community which has no existing or planned industrial development. Therefore, it is expected that wastewater from Los Osos will not have levels of toxic organics requiring additional treatment. If industries with toxic organics in their wastewaters are included in the Los Osos municipal sewerage system in the future, source control and pretreatment of industrial wastewater would be required before discharge to the municipal system.

GROUNDWATER RECHARGE REQUIREMENTS

Groundwater recharge is one of the goals of ths project. Discharge requirements for the project will therefore be projected based on the assumption that treated effluent will be used for groundwater recharge, either at Los Osos Creek (Alternative 1) or at percolation ponds (Alternative 3). The criteria used to determine the projected discharge requirements are discussed below.

TABLE 4.2

DEPARTMENT OF HEALTH SERVICES
SELECTED RECLAIMED WATER QUALITY REQUIREMENTS

Parameter	Groundwater Recharge	Landscape Irrigation	Agricultural Irrigation: Food-Spray	Agricultural Irrigation: Food-Surface	Agricultural Irrigation: Fodder, Fiber, & Seed	Agricultural Irrigation: Milking Animals Pasture
Coliform bacteria, 7-day median (MPN/100mL)	2.2[a]	23	2.2	2.2	--	23
Turbidity (NTU)	2	--	2	--	--	--
Secondary Treatment	yes	yes	yes	yes	no[b]	yes
Tertiary Treatment	yes[c]	no	yes	no	no	no

[a] Value required in the groundwater.
[b] Primary treatment required to produce effluent with not more than 0.5 ml/L/hr settleable solids only.
[c] Owing to the coliform requirement.

NOTE: Requirements are based on Title 22, Division 4, of the California Administrative code and discussions with DOHS staff.

254/13

Los Osos Creek Discharge

The tentative bacteriological standard set by the RWQCB for discharge to Los Osos Creek is 2.2 MPN/100 ml in the effluent. DOHS requirements indicate that a one-step reduction to 23 MPN/100 ml may be allowed at selected times if there is a minimum 20:1 dilution in the creek. Similarly, with a 100:1 dilution the standard may be 230 MPN/100 ml. There are, however, several inherent difficulties in administering and monitoring this type of discharge standard for an intermittent creek, particularly when surface continuity between Los Osos Creek and Morro Bay is a concern.

Percolation Ponds Discharge

DOHS and RWQCB requirements for treated wastewater discharge to percolation ponds which recharge a known potable water aquifer are determined on a case-by-case basis. The relative quantity of recharged water as compared to the total groundwater flux is a subjective criterion used to evaluate risk and resultant treatment level required.

If treated wastewater is discharged as part of a planned groundwater recharge project for aquifers used for domestic water supply, the secondary effluent must be filtered and disinfected such that the bacterial level in the receiving groundwater does not exceed 2.2 MPN/100 ml. However, the discharge may be considered simply effluent disposal and percolation of good quality secondary effluent would be acceptable if dilution at the nearest well is adequate as determined by the RWQCB and the DOHS. Adequate effluent dilution may occur if the hydrogeologic characteristics of the aquifer naturally induce dilution and/or if the distance to the nearest well is great.

Groundwater monitoring wells would be installed downgradient of the pond in order to monitor the groundwater bacteriological limit. Monitoring well locations will be determined by a hydrogeologic study to assure that the sampled water is representative of the vertical and lateral extent of the recharge plume after blending with the *in situ* groundwater.

EFFLUENT QUALITY CRITERIA AND TREATMENT REQUIREMENTS

Effluent quality criteria for the Los Osos treatment facilities have been projected based on the foregoing discussion of RWQCB and DOHS requirements and are presented in Table 4.3.

TABLE 4.3

EFFLUENT QUALITY CRITERIA
LOS OSOS WASTEWATER TREATMENT FACILITY

Parameter	Discharge Quality
BOD_5 (mg/l)	10[a]
TSS (mg/l)	10[a]
Turbidity (NTU)	2
NO_3 - N (mg/l)	5
DO (mg/l)	5[a]
Coliforms, 7-day median (MPN/100 mL)	2.2

[a]Required when discharging to Los Osos Creek only.

LANDSCAPE IRRIGATION REUSE

To achieve the effluent quality detailed in Table 4.3, the following treatment steps are required:

- Screening - to remove coarse solids
- Grit removal - to remove hard-to-handle inert grit
- Secondary treatment and nitrogen removal - to remove biodegradable organic material, suspended solids, and nitrogen
- Tertiary treatment - to remove bacteria and viruses

Three alternative methods for secondary treatment and nitrogen removal are developed and evaluated in Chapter 7. Tertiary treatment will consist of chemical addition, mixing, flocculation, filtration, chlorination, dechlorination, and final aeration.

Reuse of some of the treatment plant effluent for landscape irrigation (Alternative 4) would allow less strict quality criteria to be applied
to that portion of the flow. This flow could bypass some treatment processes, potentially allowing smaller sizing of the partially bypassed facilities and resultant cost savings. Groundwater recharge objectives would still be served in that the reclaimed water would replace groundwater which is currently pumped. However, locations of potential landscape irrigation sites (golf course, etc.) may be too distant from the treatment facility for this option to be practical. Although this possibility will be investigated further in Phase II, for purposes of this Phase I report it is assumed that no effluent will be used for landscape irrigation.

AGRICULTURAL IRRIGATION REUSE

For the purposes of this report, it is assumed that wastewater reclamation for agricultural irrigation will not affect the effluent quality criteria. Approximately 80 percent of agricultural water use in the area is for food crops (Reference 4.1). The amount of water used to irrigate pasture land (about 20 percent of area use) is too little to make it economically feasible to install pipelines to pastureland only. To allow farmers the option of spray irrigation, turbidity and bacteriological standards would be identical to those for groundwater recharge. The major difference in treatment requirements would be that nitrogen removal requirements could be significantly less stringent owing to the agricultural uptake when used for crop irrigation. For example, lettuce acreage would not require any nitrogen removal from irrigation water because nitrogen is normally added to this crop as fertilizer. However, sugar peas might require some nitrogen removal from irrigation water. This could be determined later if irrigation of sugar peas proved to be practical and desirable. This would not significantly affect capital costs, but would reduce annual energy costs.

CHAPTER 5

GEOLOGIC INVESTIGATIONS FOR DETERMINING GROUNDWATER RECHARGE

CHAPTER 5

GEOLOGIC INVESTIGATIONS FOR DETERMINING GROUNDWATER RECHARGE

INTRODUCTION

A paramount objective of the proposed CSA No. 9 sewerage project is to maximize recharge of the groundwater basin with treated effluent from the project wastewater treatment facilities. Effective groundwater recharge is dependent on a hydrogeologically suitable site which will allow percolation of effluent into the groundwater basin. This chapter discusses the hydrogeology of the Los Osos groundwater basin as it relates to groundwater recharge with treated effluent. Proposed disposal/percolation sites and criteria for evaluation of the sites are also presented.

HYDROLOGIC SETTING

Numerous investigations pertaining to the hydrology of the Los Osos hydrologic basin have been completed (see References 5.1 through 5.8). In addition, The Morro Group is presently working on a hydrologic study of the basin as part of an Environmental Impact Report for the proposed sewerage project. The U.S. Geological Survey, San Luis Obispo County Flood Control and Water Conservation District (SLOCFC&WCD), and Department of Water Resources (DWR) are in the process of a two and one-half year hydrogeology investigation of the basin. However, to date there is still no consensus of opinion regarding what the probable safe yield of the Los Osos groundwater basin is.

The selection of alternative sites for infiltration into the groundwater basin by the San Luis Obispo County Engineering Department is based on the assumption that there are two discrete aquifer zones inside and that no underflow originates outside the Basin boundaries. Based on these assumptions and the fact that the primary source of municipal water for the area is deep groundwater (lower aquifer), recharge to the lower aquifer is preferable to recharge to the upper aquifer. Extraction of groundwater from the lower aquifer is expected to increase as water demand in the area increases. In addition, evidence of salt water intrusion in the lower aquifer has been detected by DWR (Reference 5.1). Existing salt water intrusion is likely a result of greater pumping of groundwater from the lower (Paso Robles) aquifer. Recharge to the Paso Robles aquifer with treated effluent would inhibit salt water intrusion.

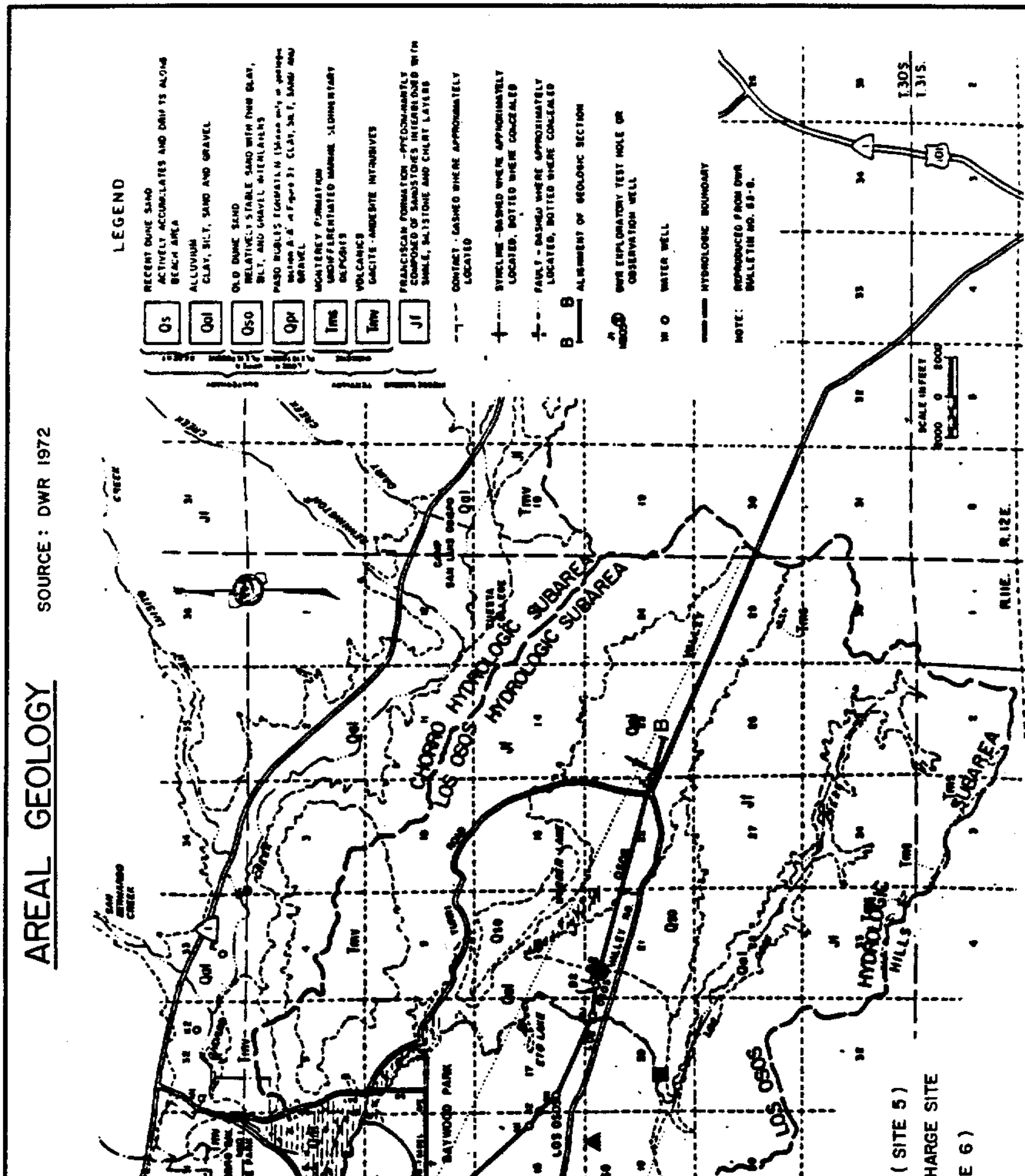
Figure 5.1 shows the geology of the area. The most prominent geologic structure of the groundwater basin is the northwest trending synclinal depression that serves to delimit the basin. Sediments from the basin margins have been deposited in the basin since Miocene time. A lithological distinction exists between sediments in the upper 100-150 feet (sand dune deposits) and those below (Paso Robles Formation). These separate units are referred to as the upper and lower aquifers. Prior investigations conclude that the upper aquifer does not extend very far to the east of Los Osos Creek.

STRATIGRAPHY

There are three primary lithologic units that comprise the Los Osos basin stratigraphy. These are, from youngest to oldest, the Old Dune sand deposits, the Paso Robles Formation, and the Franciscan Assemblage. The Franciscan Assemblage is the basement complex of the region upon which the sediments of the Paso Robles Formation and Old Dune sand deposits accumulated. The lower aquifer system is composed of the Paso Robles Formation while the upper aquifer system consists of the Old Dune sand deposits. Figure 5.1 shows the areal distribution of the geologic units in the basin while Figure 5.2 depicts a cross-section along an east-west line.

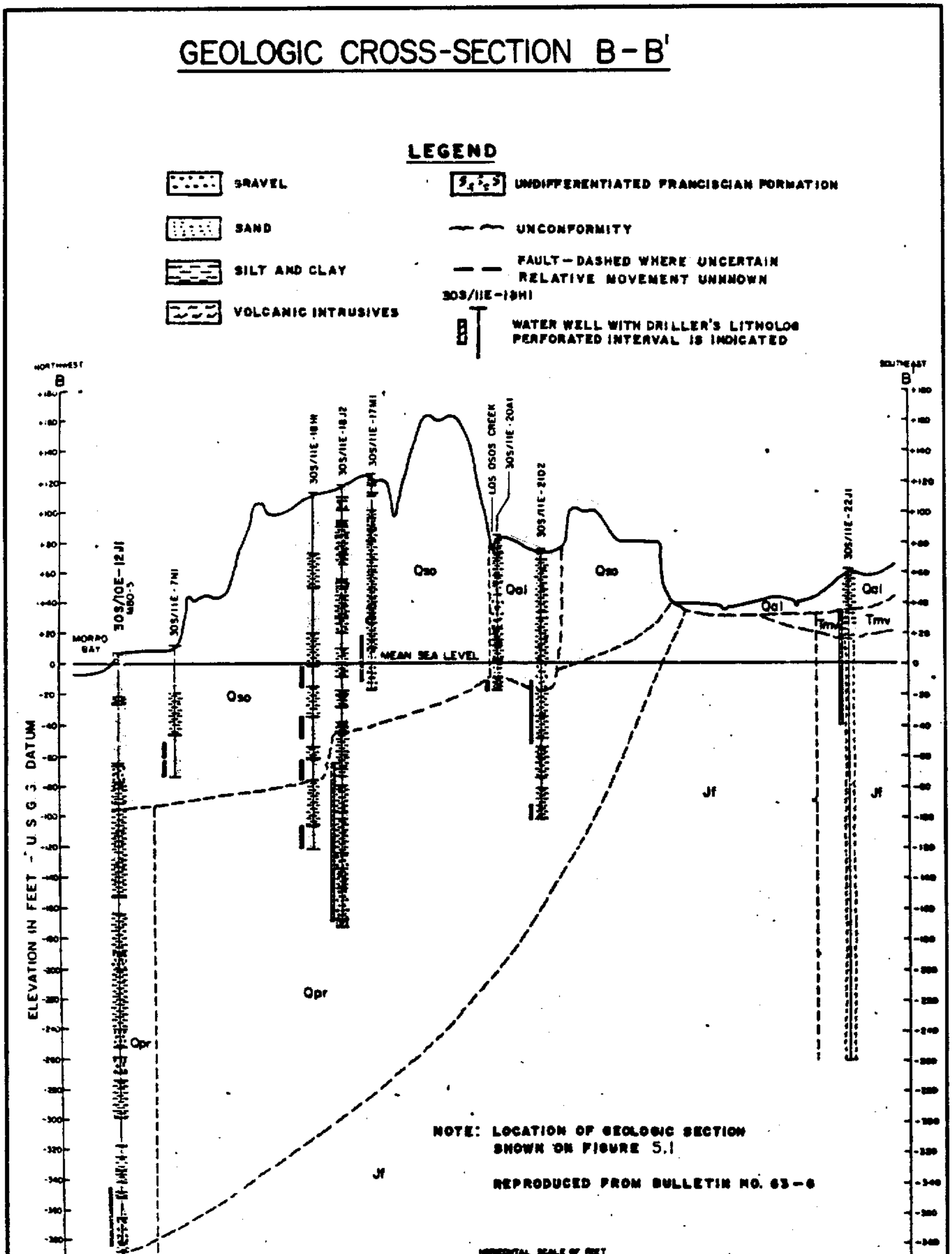
The Franciscan Assemblage represents the earliest geologic phase within the region. These rocks are predominantly composed of graywacke interbedded with shale, siltstone and chert and are not developed for water resources. The depth of the lower boundary of this assemblage is unknown but thought to be in excess of 15,000 feet below sea level.

FIGURE 5.1



ENGINEERING-SCIENCE, INC.

FIGURE 5.2



ENGINEERING-SCIENCE, INC.

Lower Pleistocene sediments of the Paso Robles Formation unconformably overlie the Franciscan Assemblage as depicted in the geologic cross-section B-B' shown in Figure 5.2. This series of sediments has been encountered in the Los Osos Valley at depths of about 100 feet below ground surface. Depths of the Paso Robles Formation range from 400 to 600 feet below sea level (Reference 5.1). These interbedded marine sediments are composed of clays, silts, sands and gravels forming the oldest water-bearing (aquifer) zone within the basin. Aquifer tests of wells perforated in the Paso Robles aquifer zone indicate permeabilities in excess of 500 gpf^2/day (Reference 5.7).

Upper Pleistocene Old Dune sand deposits are composed of very fine to medium-grained arkosic sands with thin interbeds of clay, silt, and gravel. In the Los Osos Valley along section B-B' these deposits unconformably overlie the sediments of the Paso Robles Formation (References 5.1 and 5.2). The thickness of these sediments in the Los Osos-Baywood Park area varies considerably, ranging from 70 to 150 feet (Reference 5.4).

The Old Dune sand deposits constitute an important source of groundwater in the Los Osos basin. Most domestic wells in the Baywood Park-Los Osos communities are installed in these deposits (Reference 5.2). These deposits are believed to absorb precipitation and transmit groundwater to the underlying deposits (Reference 5.6).

WATER QUALITY

The general water quality of the study area currently depends on factors including: (1) mineral or chemical character of precipitation before infiltration and surface runoff; (2) chemical character of lithologic unit through which water percolates; (3) chemical character of sewage effluent discharged to groundwater from on-site septic tank/leach field systems; (4) chemical character of irrigation runoff/infiltration waters; (5) chemical character of nearshore evaporite deposits; and (6) extent of salt water intrusion (References 5.3, 5.6, and 5.11). Once groundwater recharge with treated effluent is implemented, treated effluent quality will replace septic tank effluent-quality as a determining factor for water quality.

Surface Water

The DWR reported in 1979 that surface waters sampled at Los Osos Creek had a moderate to high (130 to 395 mg/l) concentration of TDS and a magnesium calcium bicarbonate character (References 5.2 and 5.4). Nitrate concentrations were substantially lower (2.0 to 2.5 mg/l) than those reported by Brown and Caldwell in 1983 or the RWQCB in 1984 (References 5.2 and 5.3). This suggests the increases in nitrate between 1970 and 1983 were due to increased community development. In March 1970, DWR sampled surface water springs emanating from the Old Dune sand deposits in the bog area along the southern margin of Morro Bay. These waters had a sodium-chloride character, a TDS of 137 mg/l and a nitrate concentration of 25 mg/l.

Groundwater

The principal source of groundwater in the Los Osos area is from the Paso Robles formation. The water in this aquifer reportedly has a sodium-chloride character and shows increased concentrations of nitrate moving from high to low water table elevations (Reference 5.2). Water samples from deeper wells extending into the Paso Robles Formation show a sodium-magnesium bicarbonate character and are said to be devoid of nitrates (References 5.1, 5.3, and 5.11).

Salt Water Intrusion

The Department of Water Resources found evidence of salt water in the sediment of the Paso Robles Formation directly beneath the Morro Bay sand spit in 1972, indicating salt water intrusion in the lower aquifer at that time (Reference 5.1). In 1979, the DWR, SLOCFC&WCD and the City of Morro Bay investigation of the groundwater beneath Morro Bay sand spit corroborated the DWR conclusion that sea water intrusion existed. Increased utilization of the lower aquifer to meet an increasing water demand will promote an advancing salt water intrusion. Groundwater recharge into the Paso Robles aquifer will act to form a fresh water barrier to advancing sea water intrusion and, given enough recharge, can reverse the extent of salt water intrusion. Thus, recharge of the lower aquifer with treated effluent either through direct percolation or indirectly through the upper aquifer will help mitigate salt water intrusion by providing additional recharge to that aquifer.

To date, wastewater has been recharged to the upper aquifer via septic tank leach fields. The degree to which this wastewater has also contributed to the recharge of the Paso Robles formation is not known. No evidence of high nitrate has been reported in the lower aquifer to date. However, some recharge to the lower aquifer probably occurs by leakage through the confining layer between the two aquifers.

SOURCE AND RECHARGE OF GROUNDWATER

The major source of supply to the groundwater in the Los Osos Basin is precipitation. Average rainfall at Los Osos was about 20 inches/year for the period spanning 1870 to 1984 (Reference 5.8). A period of greater than average rainfall occurred between 1964 and 1983, coincident with the period of increasing community development. Thus, long-term reliability of the groundwater resource should be viewed in the context that future recharge to the groundwater by precipitation may be more l-imited than in the recent past (Reference 5.8). The DWR has suggested that a minor amount of connate water may rise through the fractures in the Franciscan Assemblage to emanate at the ground surface as springs or mix with meteoric water below the ground surface (Reference 5.1).

A high percentage of precipitation infiltrates directly into the unconfined Old Dune sands of the Los Osos hydrologic basin. Although no values for infiltraton rates were cited, DWR believes that because of the high infiltration rates and storage capacity of these deposits, surface runoff is negligible (Reference 5.1). Septic tank effluent is also a major source of recharge for the upper aquifer system. The probable primary source of recharge to the Paso Robles aquifer zone is down-gradient infiltration from a one-mile section of Los Osos Creek (Reference 5.8). The Paso Robles formation also receives subsurface recharge from the overlying Old Dune sand aquifer system as well as minor amounts of connate water from the underlying rocks of the basement complex.

Municipal water wells in the Los Osos-Baywood Park area pump water mainly from the lower, Paso Robles aquifer. Prior to installation of municipal water systems in the 1950's, domestic water supply was obtained through numerous private wells primarily pumping water from the shallow, Old Dune sand aquifer.

MOVEMENT AND DISCHARGE OF GROUNDWATER

Water that has infiltrated the subsurface materials moves from areas of high hydraulic head to areas of low hydraulic head. Groundwater moves down the hydraulic gradient and discharges to the lowest water level elevation of an aquifer system. generally, groundwater flow in the los osos basin is westerly toward the ocean. both dwr and the morro group have presented independent data for the upper aquifer zone for the years 1970 and 1984 which verify the westward flow direction (references 5.1 and 5.8). east of the eastern hydrologic boundary of the los osos groundwater basin, groundwater flows to the east in the upper formation.

The California Regional Water Quality Control Board (RWQCB), using water quality data of the San Luis Obispo County Engineering Department, came to the conclusion that the groundwater gradient in the Los Osos-Baywood Park area generally conformed to the surface topography (References 5.10 and 5.11).

Excess groundwater in the Los Osos basin emanates as springs and seeps in the Old Dune sand deposits. These springs and seeps form a bog along the southern margin of Morro Bay. Water from these sands probably also discharges beneath the Bay where the aquifer intersects the Bay floor.

Water Budget

In their 1983 Water Quality Study, Brown and Caldwell stated that the safe yield of the Los Osos hydrologic basin was on the order of 1300 to 1800 acre feet per year (Reference 5.2). Brown and Caldwell also contended that the return-water factor for domestic use in the form of septic tank effluent was 58 percent of the total water use (Reference 5.2). The DWR estimated that the total volume of subsurface water outflow greatly exceeded withdrawals from water wells (Reference 5.1). The basis for this 1972 contention was derived from the fact that sea water intrusion was very limited along the coastal margins.

EFFLUENT DISPOSAL/PERCOLATION SITE SELECTION

In 1985, the Morro Group examined six potential sites for treated effluent percolation ponds to recharge the lower aquifer. The sites were considered based on the assumption that the upper aquifer does not extend east of Los Osos Creek. Of the six sites examined by the Morro Group, three were selected as warranting additional investigation. These are: the Cemetery Mesa site, located in an area where rocks from the Paso Robles Formation were thought to outcrop; the upper Los Osos Creek site, about one mile above the Los Osos Valley Road bridge; and the Broderson site, located south of Highland Road. Geotechnical and hydrologic investigations have occurred at the Cemetary Mesa Site and Broderson Road site; site specific investigations still remains to be done at the Los Osos Creek site. These sites are shown on Figure 5.1.

A detailed investigation of the Cemetary Mesa site revealed that it was unsuitable for percolation due to a thick upper layer of relatively impermeable clays and silts which could belong to either the Paso Robles Formation or the Old Dune sand deposits. The Cemetary Mesa Site was therefore eliminated as a viable recharge site.

With no other feasible sites available for direct recharge to the lower aquifer from percolation ponds, the County decided to investigate utilizing Los Osos Creek for groundwater recharge. Due to the potential inability of Los Osos Creek sediments to absorb treated effluent during wet weather periods and to the probable RWQCB prohibition of treated effluent discharge whenever surface water continuity exists between the discharge point and Morro Bay (refer to Chapter 4 and Appendix D), it was decided that the Los Osos Creek site should be used during the dry season only.

The Broderson Site (Site 6) was selected to be investigated as a potential site for wet weather recharge to the upper and lower aquifers via percolation ponds. Geological, geophysical and seismic data from this site are discussed later in this chapter.

CRITERIA FOR EVALUATION OF DISPOSAL/PERCOLATION SITES

General

Percolate flow in the unsaturated zone beneath a recharge basin is essentially vertical. If a hydrologic barrier exists at depth, a horizontal component of flow is introduced and flow is controlled by both vertical and horizontal permeabilities (References 5.12 and 5.13). The capacity of lateral flow away from the potential percolation sites controls the extent of groundwater mounding that will occur beneath the site (Reference 5.11). The travel time for lateral flow is a function of the hydraulic gradient, the distance of travel, and the horizontal and vertical permeability (Reference 5.14). In many cases the percolate emerges as base flow in adjacent surface waters. Often seeps or springs may develop in intervening terrain.

No geotechnical investigations have been completed for the Los Osos Creek recharge site to date. However, the necessity of detailed investigation at this site is less pertinent as existing evidence indicates that sediments in Los Osos Creek provide a medium for infiltration which eventually recharges the Paso Robles Formation.

Soil investigations developed by Pacific Geoscience, Inc. under the direction of the Morro Group have been completed for the Broderson Road percolation pond site. The scope of these investigations includes field and laboratory testing and observations based on borings and seismic refraction study. Further discussion of investigation at the site-specific level is presented later in this chapter.

Site Suitability

Evaluation of site suitability for proposed areas of recharge must consider the objectives of recharge, site hydrogeologic characteristics, geologic hazards associated with the site (seismic shaking, landslide hazards, flooding, liquefaction), surface permeability and aquifer recharge geometry. Special emphasis should also be given to site topography and soil type and uniformity.

The primary objective of groundwater recharge for this project is to recharge the lower aquifer for purposes of water supply augmentation and salt water intrusion mitigation, as discussed earlier in this chapter. The secondary objective is to recharge the upper aquifer which may allow indirect recharge of the lower aquifer. The selection of the the Los Osos Creek Site was based on the objective of recharging the lower, Paso Robles Formation aquifer. The selection of the Broderson Site was based on the objective of recharging the upper aquifer system which would then infiltrate into the Paso Robles Formation during the winter months when discharge to Los Osos Creek would not be permitted.

Hydrogeologic Characteristics

Los Osos Creek Site. Site specific hydrogeologic conditions of the Los Osos Creek recharge site have not been determined. Additional geotechnical investigations required to assess baseline conditions more definitely at this site are recommended in Chapter 8.

Broderson Avenue Site. Seven boreholes were drilled by Pacific Geoscience, Inc. under the direction of the Morro Group to delineate the geologic characteristics of the site. The first unit encountered in all of the boreholes is the the wind-blown sand which consisted of poorly consolidated, fine sand with relatively minor amounts of silt and essentially no clay. The Morro Group suggested that three of the boreholes (boreholes 1, 2, and 5) drilled on the site displayed the presence of moderately consolidated sandstones, clayey siltstones, and silty claystones indicative of the Paso Robles Formation; however, lithologic differences between the Paso Robles and overlying sands based on the borehole data are difficult to distinguish.

Twenty-nine seismic refraction profiles shot at or adjacent to the site supplemented the borehole data and assisted in distinguishing between the two formations based on the sharp change in seismic velocity (1100-1300 ft/sec for the wind-blown sands as opposed to 2000-3000 ft/sec for the Paso Robles Formation). These data indicate that the wind-blown sand unit is thicker (15-40 feet) beneath the western and northern portions of the site, and thinner (0-15 feet) beneath the southeastern section of the site. The field studies also confirmed that the Paso Robles Formation, unconformably underlying the wind-blown sands, dips toward the northwest at approximately 10 degrees in outcrops located southeast of the site (Reference 5.15).

Appendix H contains a letter report from the Morro Group which summarizes the work of Pacific Geoscience, shows location of boreholes and seismic refraction profiles, plots thickness contours for the wind-blown sand unit, presents permeability ranges for the two units, and suggests loading rates and percolation basin geometry. Additional geotechnical and hydrogeological investigations required to determine hydraulic loading parameters and baseline conditions are recommended in Chapter 8.

Geologic Hazards

Los Osos Creek Site. The Los Osos Creek Site lies in an area of moderate-to-high liquefaction potential and negligible landslide risk (Reference 5.5). Very little preliminary work has been done regarding this candidate site. A more detailed geotechnical investigation of the site is needed to fully assess its suitability.

Broderson Avenue Site. The Broderson Avenue Site lies outside the floodplain but is located in an area of moderate liquefaction potential (Reference 5.5). Liquefaction occurs when a loosely packed, granular sediment (the wind-blown sand unit) is transformed into a fluid mass due to an increase in pore pressure or reduction in effective stress.

Surface Permeability

Los Osos Creek Site. No known data are currently available on the permeability of the surface materials at the Los Osos Creek Site. The recently installed weir approximately one-half mile upstream of the bridge along Los Osos Creek can provide important data on recharge capabilities of the sediments; as stream flow diminishes into the dry season, observations can be made concerning the role of upstream disappearance of the surface expression of the stream. This would provide a qualitative idea of the infiltration. A quantifiable evaluation of the permeability characteristics of the streambed sediments will require field permeability tests during the dry season of no stream flow.

Broderson Avenue Site. Permeability or infiltration rate tests for the site were conducted by Pacific Geoscience during the fall of 1985. In each of the seven boreholes, percolation rates at depths of 10 and 25 feet were determined. At a depth of 10 feet, the percolation rate in the wind-blown sand ranged from 0.1 to 2.7 minutes/inch, whereas, at 25 feet depths the percolation rate in the same unit ranged from 0.1 to 5.0 minutes/inch. Based on these data, Pacific Geoscience suggested a permeability of 3 minutes/inch parallel to the bedding in the wind-blown sands (see Appendix H).

Only one percolation test was performed on the Paso Robles Formation. This test resulted in a precolation rate of 13 minutes/inch at a depth of 25 feet. Comparision of this result with 25 foot depth test results for the wind blown sand shows that percolation in the Paso Robles Formation is between 10 to 100 times slower than in the wind-blown sands. These differences in aquifer characteristics must be factored into the design of the percolation basins.

Aquifer Recharge Geometry

Los Osos Creek Site. The Morro Group reported that, based on shallow groundwater levels and their changes over time, the upper aquifer zone is recharged by infiltration of rainfall, septic tank effluent, and excess landscape irrigation (Reference 5.8). Recharge of the lower aquifer zone appears to be relatively unaffected by increase of aquifer head in the shallow aquifer, suggesting that the probable primary source of recharge of this aquifer is from northwesterly, down-gradient infiltration from Los Osos Creek. The most effective section of recharge is believed to be a one-mile portion of the Creek situated upstream from the Los Osos Valley Road Bridge (Reference 5.8). Treated effluent discharged at the Los Osos Creek Site should recharge the lower (Paso Robles) aquifer zone.

Broderson Avenue Site. Examination of borehole logs and seismic refraction and infiltration test data suggests that the Broderson Avenue Site will recharge the lower aquifer system (Paso Robles Formation). The design geometry of the recharge basins is depicted and discussed in Chapter 8 of this report. Hydrogeologic factors which influenced design considerations were: 1) the thickness of the dune sands overlying the Paso Robles; and 2) the estimated permeability of the Paso Robles Formation.

Wastewater percolating through the wind-blown sands will flow downslope and gradually infiltrate downward into the Paso Robles Formation. The Morro Group estimates that all of the wastewater used for recharge would infiltrate into the Paso Robles Formation within 600 feet of the infiltration basins. This estimate is based on the Morro Group's infiltration test results at the site. Even if the wind-blown sands thin at some distance downslope, which seems unlikely given the apparent thickening of the deposit in a northerly direction, there appears to be at least a 10-foot thickness of wind-blown sands. Based on the estimates provided by the Morro Group, this thickness should provide an adequate cover to prevent any surfacing of percolated wastewater downslope so long as the head on the Paso Robles formation remains at or below 6.7 feet (see Appendix H).

Seismic refraction data reveal that groundwater elevations in the Paso Robles Formation are between 117 and 150 feet below the top of the unit. Hence, the depth to groundwater is not an influencing criterion in the location or the design of the recharge basins. However, the depth to groundwater is an important factor in determining the location and depth of water quality monitoring wells.

CHAPTER 6

DEVELOPMENT AND EVALUATION OF ALTERNATIVE COLLECTION SYSTEMS

CHAPTER 6

DEVELOPMENT AND EVALUATION OF ALTERNATIVE COLLECTION SYSTEMS

INTRODUCTION

Initial cost estimates for construction of a wastewater collection, treatment and disposal system to serve the Los Osos area indicate that the cost of the system to users will be high. A re-evaluation of the proposed project has been undertaken to determine if feasible alternatives are available to reduce to a substantial degree the cost of the sewerage system.

The preliminary capital cost estimate for construction of a conventional sewage collection system proposed for the service area represents nearly 80 percent of the total estimated cost of the overall system (Reference 6.2). If a significant cost reduction is to be achieved in the cost of the overall system, it must therefore be realized primarily in the cost of the collection system. The fact that the collection system cost is approximately 80 percent of the overall project is not unusual for an existing community which is currently not sewered. Other communities faced with high costs of providing a sewage collection system have turned to "alternative" collection systems to realize significant cost savings over conventional sewer systems. These cost savings have usually resulted from the ability of alternative systems to accommodate conditions which are adverse to conventional gravity sewers such as sparse population, hilly terrain, high groundwater and shallow bedrock. The goal of alternative systems is to perform the same function as conventional gravity sewers at a more affordable cost.

The Los Osos project is a potential candidate for the application of alternative collection systems because of high groundwater and adverse topography in some of the service area. A factor which favors conventional gravity sewers for the project, however, is the very high residential density and relatively large population of most of the service area compared to communities which have historically utilized alternative collection systems.

This chapter will evaluate viable alternative sewage collection systems and compare them to a conventional gravity sewer system. The primary basis of this comparison will be cost, including capital and operation and maintenance (O&M) costs, although other factors must and will be considered. Four types of alternative collection systems will be evaluated. The traditional conventional gravity sewer system will be used to set standards of comparison in terms of cost and performance. The first two types of alternative systems are two pressure sewer system variations: (1) septic tank effluent pumping and (2) grinder pumping. The third alternative system consists of variable-grade gravity sewers which have recently emerged as a feasible alternative system for small communities. The fourth alternative system which will be considered is a hybrid system incorporating all or some of the other alternatives with the goal of providing the least costly system.

Alternative systems will first be presented and qualitatively evaluated in a general manner. Conceptual level design of systems to serve the Los Osos area will then be developed as appropriate. Based on the conceptual design, capital and operation and maintenance costs will be estimated and a long-term present worth economic analysis will be performed to determine the least costly alternative. A final evaluation will incorporate the cost evaluation and other factors to produce a recommended collection system. The evaluation will determine if the use of alternative systems in whole or in part can substantially reduce the cost of the sewage collection system.

CONVENTIONAL GRAVITY SEWER SYSTEM

Conventional Gravity Sewer System Description

A conventional gravity sewer system is the traditional solution to the problem of environmentally acceptable sewage collection. Conventional gravity sewers are used as underground open channels. Gravity sewers rarely flow full and are designed to have a constantly downward slope to maintain an open channel flow regime and to prevent low spots which induce solids deposition. Gravity sewer slopes must be adequate to maintain a flow velocity that will prevent solids deposition and sewage septicity. Sewage from individual users drains through service lateral sewers to a collector sewer where it flows to a larger interceptor sewer and subsequently to the treatment facilities. If the topography of the area is such that gravity flow from source to treatment facilities is not possible, sewage pump stations and force mains are used to transport sewage uphill or across long, relatively flat reaches.

The profile of a gravity sewer must maintain its constantly downward slope regardless of ground profile. To minimize deep excavations for sewer construction and thereby minimize construction costs, gravity sewers are designed to generally correspond to the natural drainage characteristics of the service area. If the destination of the sewage is not the low point of the drainage basin, or if the sewered area contains several drainage basins, a pump station and force main are used to transport sewage out of the drainage basin.

Gravity sewers are designed to be of a size and slope adequate to accommodate peak flows including infiltration and inflow (I/I). Infiltration is groundwater which leaks into sewers while inflow is direct rainfall runoff that drains into the sewer system. Manholes are provided on conventional gravity sewers to provide access for O&M activities such as cleaning, inspection and repair. Manholes are spaced at minimum intervals and are installed at changes in sewer size, alignment or grade. Gravity sewers have constant size, slope and alignment between manholes to facilitate maintenance.

Sewage pump stations are designed to have adequate capacity to pump expected peak flows including I/I. Provisions for standby power and back-up pumps and other equipment are included in pump station design to insure reliability and prevent sewage overflows. Due to the amount of mechanical equipment involved with a sewage pump station, maintenance is an important consideration. Sewage pump stations are designed to operate automatically and are unmanned except during routine maintenance. Sewage pump stations should include a remote monitoring system to detect problems at the station so they can be corrected in a timely manner.

Sewage force mains are equipped with automatic air release valves to prevent the accumulation of air at high points in the pipeline. The build-up of air at high points would tend to throttle the force main and restrict its hydraulic capacity. Vacuum breaker valves are also provided at some force main high points to prevent water column separation and vacuum conditions in the pipeline which could induce damaging hydraulic transient forces. Equipment to mitigate hydraulic transients in the force main may be required at the pump station.

Conventional Gravity Sewer System Technical Feasibility

The technology of gravity sewers is quite simple and has been used successfully for many years. Gravity sewers are relatively easy to design and there are many contractors who have extensive experience in gravity sewer construction. The technology of sewage pump stations is not quite as simple as that of gravity sewers, but it is just as well-proven. Recent technological advances in control and monitoring systems for sewage pump stations have served to reduce O&M costs and to provide more reliable, automatic pump stations. Many established manufacturers of sewage pumps and related equipment are available.

Conventional Gravity Sewer System Advantages

Assuming a reasonable total number of pump stations will be required in a sewer system, the primary advantage of conventional systems over alternative systems is low O&M costs. Maintenance for gravity sewers themselves basically consists of routine cleaning. Remaining O&M requirements for gravity sewer systems relate to sewage pump stations and force mains. Another positive consideration of conventional systems is that they have proven long-term reliability and present little risk of presenting future unexpected or extraordinary maintenance, repair or replacement costs as may the alternative systems. Based on their long history of reliable performance, gravity sewer systems can be expected to have a very long useful life, especially considering modern materials and contruction techniques. Conventional gravity sewer systems therefore offer low maintenance, a proven track record and a long life expectancy.

Conventional Gravity Sewer System Disadvantages

In certain circumstances, conventional gravity sewer systems may not be economical in terms of cost per user. Conditions which could make gravity sewers uneconomical include sparse population, rugged or hilly terrain, high groundwater or shallow bedrock. A conventional sewer system which requires numerous pump stations or deep sewers may prove to be more costly than an alternative system. A disadvantage of conventional gravity sewers is that the sewers must maintain a constantly downward slope. This constraint can cause construction difficulties relating to utility interferences, high groundwater, soil geology and deep excavation.

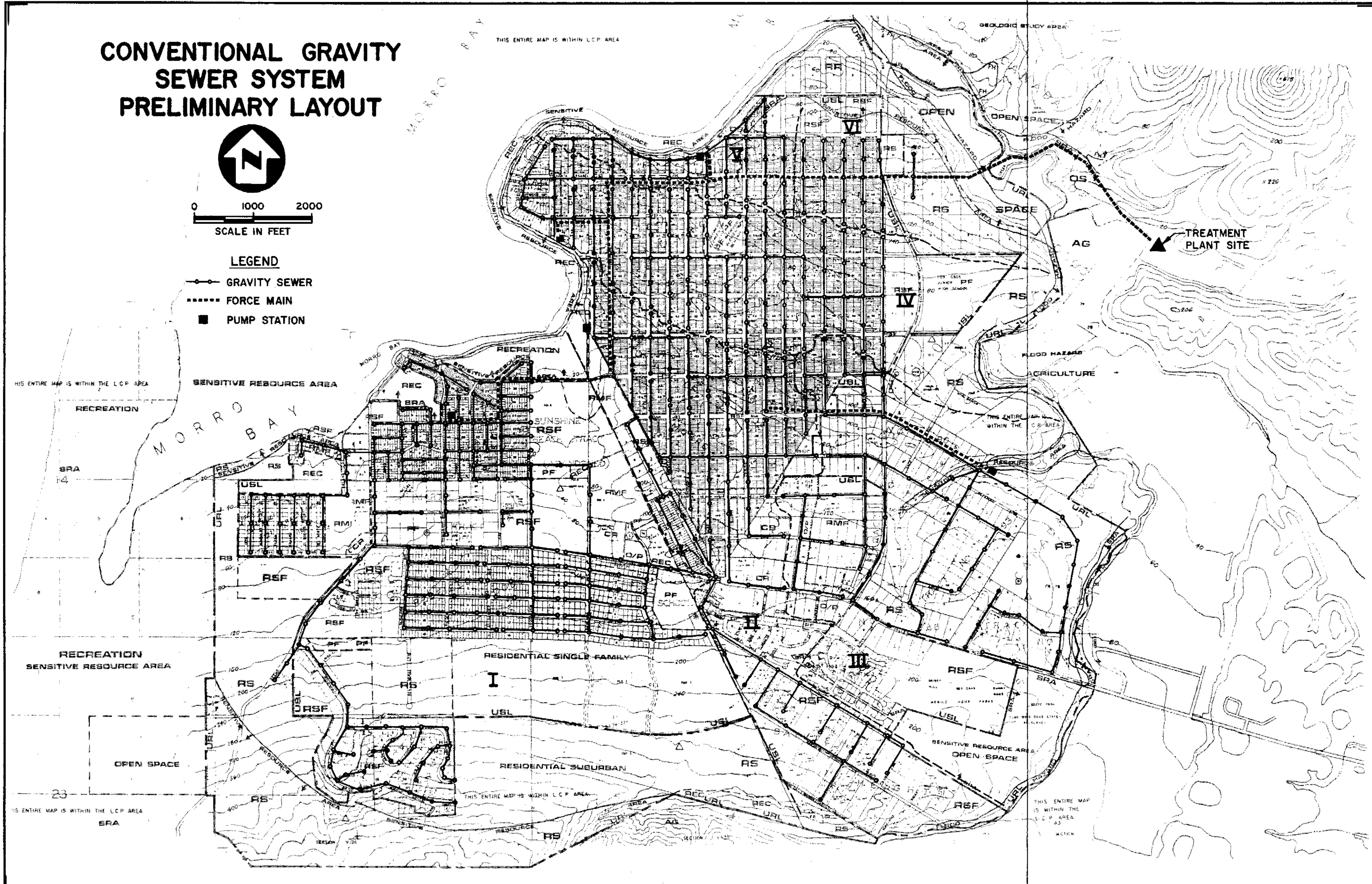
I/I in conventional gravity sewer systems is an undesirable yet usually unavoidable system component. As sewers increase in age, I/I will increase and may become excessive to the point where pumping and treatment capacity is constrained. I/I is a definite disadvantage of conventional gravity sewers, but can be mitigated through high-quality pipe materials and construction and an ongoing preventative maintenance program.

Conventional Gravity Sewer System Conceptual Design

A preliminary layout of of a conventional gravity sewer system to serve the Los Osos area has been developed and is shown in Figure 6.1. Gravity sewers are provided to serve all existing lots of record in the service area and to accommodate future development of currently unsubdivided land. Gravity sewer preliminary sizes have been selected based on unit flow rates from Chapter 3 and on assumed minimum slopes. The service area was divided into six sewer drainage basins based on USGS topographical data. Five of the six drainage basins will require a sewage pump station. Pertinent information regarding the six gravity sewer drainage basins is given in Table 6.1.

The topography of most of Basin I slopes consistently downward toward the ocean and is very favorable for gravity flow sewers. The area of Basin I between the 20 foot contour and the shoreline, however, is very flat and is expected to have high groundwater. This flat area will require deep sewers which will likely be below the groundwater

FIGURE 6.1



ENGINEERING-SCIENCE, INC.

TABLE 6.1

CONVENTIONAL GRAVITY SEWER SYSTEM
DRAINAGE BASIN CHARACTERISTICS

Basin	25' Wide Lots	Other Lots	Total Lots	Sewered[a,b] Lots	Mobile[c,d] Homes	Area (acres)
I	310	1,940	2,250	68	164	1,460
II	1,890	310	2,200	111	--	480
III	270	240	510	123	450	610
IV	2,480	100	2,580	--	--	450
V	580	20	600	--	--	140
VI	470	10	480	--	--	210
Total	6,000	2,620	8,620	302	614	3,360

[a]Sewered lots are all greater than 25' wide.
[b]Included in Total Lots.
[c]Mobile homes are sewered.
[d]Not included in Total Lots.

table. Basin I is the largest of the sewer drainage basins and encompasses most of the undeveloped land within the service area. Nearly all of this currently undeveloped area may be developed in accordance with the County Land Use Plan, primarily for single family and multiple family dwelling units. Existing development within Basin I consists almost entirely of residential subdivisions including Cuesta-by-the-Sea, Morro Shores Mobile Home Park, Sunset Area, Upland Area, Vista de Oro and Cabrillo Heights. Vista de Oro subdivision and Morro Shores Mobile Home Park each have existing gravity sewer collection systems which discharge to community septic tank/leach field systems.

Basin II encompasses the southern portion of the high density residential area of Baywood Park, the west half of the commercial area of Los Osos Village, the low density residential area of Bayview Heights and the west half of the Bayridge Estates subdivision. The terrain of Basin II varies. Topography of Bayview Heights area is similar to Basin I and is favorable for gravity sewers. The southern tip of the Bayview

Heights area is within the drainage basin of Los Osos Creek. The few houses in this area would require on-lot pumping facilities to connect to the gravity sewer system. The topography of most of the rest of Basin II is undulating and erratic, representative of wind-blown dune formations. The drainage of the area of Basin II north of Los Osos Valley Road consists of many small basins which drain to the Bay, to Los Osos Creek, or to low lying areas which must be pumped or drained. The area between the 100 foot contour and the 120 foot contour exhibits no particular drainage pattern. The topography of this area is not conducive to gravity flow sewers and will result in a significant amount of deep sewer cuts. The Bayridge Estates subdivision has an existing gravity sewer collection system which currently discharges to a community septic tank/leach field system. The existing gravity sewer system which serves the west half of Bayridge Estates will drain to Basin II.

Basin III is within the drainage basin of Los Osos Creek which flows to the north of the service area. Basin III encompasses the low density residential Creekside area, the area of Los Osos Village not in Basin II, a small portion of Baywood Park, the east half of Bayridge estates and three mobile home parks. Much of the Creekside area is currently undeveloped. The divide between Basin II and Basin III north of Los Osos Valley Road is in the area of undulating terrain between the 100 and 120 foot contours and is therefore not well defined. The topography of the remainder of Basin III slopes downward to the center of the basin to a tributary of Los Osos Creek. The east half of Bayridge Estates has an existing gravity sewer system which discharges to the community septic tank and leach field. Each of the three mobile home parks also has a community collection system and septic tank.

Basin IV lies in the central portion of Baywood Park between two distinct ridge lines. The ridge to the south separates Basin IV from Basin II and the ridge to the north separates Basin IV from Basin V and Basin VI. Topography of Basin IV is generally downward away from the ridges and to a central low area. Although most of the area between the ridges drains toward the Bay, the inland portion tends to drain toward Los Osos Creek. The low area between the ridges does not drain well, particularly below the 40 foot contour towards the Bay and around the 80 foot contour towards Los Osos Creek. This low area experiences ponding during wet weather and has groundwater very close to the ground surface. The flat, low areas of Basin IV will require deep sewers which will likely be below the groundwater table. Development within Basin IV is predominantly high-density, single-family residential and includes an elementary school and junior high school. Areas zoned for commercial and multi-family development also exist within Basin IV.

Basin V lies between Morro Bay and a ridge which separates the area from the Basin IV. The terrain slopes relatively steeply from the ridge to the shoreline. While the slope is conducive to gravity sewers, the flat shoreline is not. Development within Basin V is typical of the Baywood Park high-density residential development although much of the subdivided area remains undeveloped. High groundwater is not anticipated in this basin except along the shoreline.

Basin VI also lies between two longitudinal ridges and is part of Baywood Park. This area is included as a separate basin because it slopes toward Los Osos Creek and can therefore drain directly to the treatment facilities without pumping.

With the exception of Basin VI, each of the sewer drainage basins will require a pump station to transport sewage to a point where it may flow by gravity to the treatment facilities. Basin III sewage will be pumped into Basin II. Sewage from Basin I, Basin II (including Basin III), Basin IV and Basin V will be pumped through a common force main interceptor to Basin VI where it will flow by gravity through an outfall-type interceptor to the treatment facilities.

Design criteria for gravity sewers and force mains are given in Table 6.2. Design criteria for the five sewage pump stations are given in Table 6.3. Only those criteria which will have a bearing on preliminary cost estimates are included. Pump stations, force mains and gravity sewers have been sized to accommodate the projected ultimate peak wet weather flow (PWWF). PWWF, per capita flow rates and per unit flow rates are developed in Chapter 3. In sizing sewers in the Baywood Park and Cuesta-by-the-Sea areas which have substandard sized lots (25-foot width), it was assumed that the ultimate number of single-family dwelling units would be 75 percent of the number of lots to account for the construction of one house on two or more lots.

TABLE 6.2

CONVENTIONAL GRAVITY SEWER SYSTEM
PIPELINE PRELIMINARY DESIGN CRITERIA

Gravity Sewers	
Design Flow	Ultimate peak wet weather flow
Minimum collector sewer size	6-inch diameter
Typical lateral size	4-inch diameter
Minimum depth of cover	3 feet
Collector sewer pipe material	PVC
Lateral pipe material	PVC or ABS
Gravity interceptor pipe material	VCP
Mannings "n"	0.009 PVC 0.013 VCP
Minimum slope	0.005
Maximum manhole spacing	400 feet
Force Main	
Design flow	Ultimate peak wet weather flow
Minimum pipe size	4 inches
Pipe material	PVC
Minimum depth of cover	3 feet
Hazen-Williams "C"	140

TABLE 6.3

CONVENTIONAL GRAVITY SEWER SYSTEM
PUMP STATION PRELIMINARY DESIGN CRITERIA

Pump Station Basin	Design[a] PWWF mgd	Design[a] PWWF gpm	Design TDH (ft)	Total HP Reqd.[b]	Standby Power	Pump Type	Pump Speed Control	Odor Control Equip.
I	2.3	1,600	145	90	permanent	dry pit	variable	yes
II	1.9	1,300	125	60	permanent	dry pit	variable	yes
III	0.8	580	115	25	portable	submersible	constant	no
IV	1.0	720	125	35	permanent	dry pit	variable	yes
V	0.3	200	120	10	portable	submersible	constant	no

[a]Ultimate flow.
[b]Assuming 70 percent overall efficiency.

Sizing of the facilities for future development was based on the zoning of unsubdivided land within the area on a rough basis. A thorough review of development plans for the area may be necessary to accurately determine the required capacities of each pump station. Although not considered in this analysis, phased construction of pumping equipment of the proposed pump stations could prove to be cost-effective, particularly for Basin I which has the greatest potential for future growth.

Final location of sewage pump stations and alignment and size of force main interceptors may differ from the locations shown on the preliminary layout. A change of location for the pump stations or force mains should not affect the outcome of this analysis. Final design of the gravity sewers will also yield modifications to the preliminary layout sizes and locations. However, a review of the layout indicates that it is acceptable and more than adequate for this study.

Conventional Gravity Sewer System Unit Costs

Gravity sewer unit costs include costs for the following items:

- Pavement saw cutting
- Trench excavation
- Shoring, bracing or other safety measures for trench excavations over five feet
- Pipe bedding material and placement
- Pipe material and installation
- Pipe zone backfill material, placement and compaction
- Backfilling and compacting remainder of trench with native material
- Water and gas service connection repair
- Pavement repair and replacement
- Manholes spaced at an average interval of 300 feet
- Spoils removal
- Testing and cleanup

As discussed above, the topography of much of the service area is quite suitable for gravity sewers. It will be possible to install gravity sewers in these areas at relatively shallow depths, above the groundwater table. The topography of other parts of the service area will dictate deep gravity sewer installations below the groundwater table. Costs for deep sewers are higher than shallow sewers because of increased excavation costs, backfill costs, dewatering costs, and additional shoring costs. In general, the progress of the work is much slower for deep sewers than for shallow sewers, thus decreasing production rates and increasing costs.

In order to attain an average unit cost for gravity sewers which would be appropriate for the Los Osos area, two unit costs were developed for each pipe size. One cost is for shallow sewers installed at an average depth of six feet with no dewatering of the trench. The second cost is for deep sewers installed at an average depth of 12 feet, requiring shoring and dewatering. The preliminary layout of the gravity sewer system was reviewed to locate and identify high cost sewers which were generally found in the flat areas along the shoreline or in the undulating terrain areas of Baywood Park and Los Osos. Separate takeoffs were performed for shallow sewers and deep sewers, and are given in Table 6.4 by drainage basin. The two unit costs given in Table 6.4 were applied to the separate totals to develop a total cost and an average unit cost.

Segregation of the gravity sewers into low-cost and high-cost categories is useful in locating problem areas in the system which could be mitigated by the use of alternative systems. This approach will be discussed further in the subsequent section of this chapter dealing with the combination system alternative.

Unit cost for sewer service laterals includes costs for the following items:

- Pavement saw cutting
- Connection wye on sewer main
- Trench excavation
- Pipe material and installation
- Backfilling of trench
- Pavement repair and replacement
- Testing and clean-up

TABLE 6.4

CONVENTIONAL GRAVITY SEWER SYSTEM
GRAVITY SEWER PIPELINE AND UNIT COST SUMMARY

Basin	6-inch Diameter			8-inch Diameter		
	Type A[a] (lf)	Type B[b] (lf)	Total (lf)	Type A[a] (lf)	Type B[b] (lf)	Total (lf)
I	61,000	8,300	69,300	13,500	1,800	15,300
II	29,000	11,000	40,000	5,100	1,900	7,000
III	25,300	9,800	35,100	600	300	900
IV	29,300	10,300	39,600	2,200	800	3,000
V	10,100	3,400	13,500	2,000	700	2,700
VI	7,100	800	7,900	--	--	--
Total	161,800	43,600	205,400	23,400	5,500	28,900
Unit Cost ($)	24.00/lf	46.00/lf	29.00/lf	25.00/lf	47.00/lf	30.00/lf

Basin	10-inch Diameter			12-inch Diameter		
	Type A[a] (lf)	Type B[b] (lf)	Total (lf)	Type A[a] (lf)	Type B[b] (lf)	Total (lf)
I	2,200	1,700	3,900	--	--	--
II	--	1,300	1,300	--	1,400	1,400
III	--	--	--	--	--	--
IV	200	4,000	4,200	--	--	--
V	--	--	--	--	--	--
VI	--	--	--	--	--	--
Total	2,400	7,000	9,400	--	1,400	1,400
Unit Cost ($)	26.00/lf	48.00/lf	43.00/lf	28.00/lf	50.00/lf	50.00/lf

[a]Type A sewers have an average depth to invert of 6 feet and require no dewatering.
[b]Type B sewers have an average depth to invert of 12 feet and require dewatering.

Service laterals will be provided to each subdivided lot. The number of laterals serving substandard size lots (25 feet wide) was assumed to be 75 percent of the total number of lots. There are approximately 6,000 substandard lots and approximately 3,000 other lots in the service area. Lots in areas with existing sewer systems were excluded in determining total lateral sewer length. Service laterals will be installed to the property line of each existing undeveloped lot. Service laterals for existing developed lots were assumed to extend 20 feet past the property line. Costs for connection to the service lateral and abandonment of existing septic tanks are not included.

Unit costs for force mains include costs for the following items:

- Pavement saw cutting
- Trench excavation
- Pipe bedding material and placement
- Pipe material and installation
- Pipe zone backfill material, placement and compaction
- Backfilling and compacting remainder of trench with native material
- Water and gas service connection repair
- Pavement repair and replacement
- Air release and air vacuum valves as required
- Spoils removal
- Testing and cleanup

Costs for the pump stations were determined based on their performance requirements as compared with the performance requirements and costs of similar pump stations. Costs for all pump stations include consideration of sheet piling and dewatering which will be necessary at each station. Design criteria for the pump stations are given in Table 6.3.

A summary of the conceptual design components of the conventional gravity sewer system and the cost estimate for the system are given in Table 6.5.

TABLE 6.5

CONVENTIONAL GRAVITY SEWER SYSTEM - PRELIMINARY CAPITAL COST ESTIMATE

Item	Quantity (lf)	Unit Cost ($/lf)	Total Cost ($)
Service Laterals			
4-inch diameter	300,000	15.00	4,500,000
Gravity Sewers			
6-inch diameter	205,000	29.00	5,945,000
8-inch diameter	29,000	30.00	870,000
10-inch diameter	10,000	43.00	430,000
12-inch diameter	2,000	50.00	100,000
Force Main Interceptor Sewers			
4-inch diameter	500	17.00	9,000
8-inch diameter	5,300	23.00	122,000
10-inch diameter	400	28.00	11,000
12-inch diameter	4,200	33.00	139,000
16-inch diameter	2,000	39.00	78,000
18-inch diameter	3,900	45.00	178,000
Gravity Main Interceptor Sewer			
21-inch diameter	8,700	60.00	522,000
Sewage Pump Stations			
Basin I Pump Station	--	--	700,000
Basin II Pump Station	--	--	500,000
Basin III Pump Station	--	--	300,000
Basin IV Pump Station	--	--	400,000
Basin V Pump Station	--	--	100,000
SUBTOTAL			14,904,000
Contingency (20%)			2,981,000
SUBTOTAL			17,885,000
Contractor's Overhead and Profit (15%)			2,683,000
SUBTOTAL (Construction Costs)			20,568,000
Technical Services			
Basic Design Services (5.7%)			1,172,000
Right-of-Way Easement Acquisition Services (2%)			411,000
Other Technical Services (12%)[a]			2,468,000
TOTAL			24,619,000

[a]Includes geotechnical, surveying, construction management, legal, financial and administrative services.
NOTE: ENR CCI 5180

Conventional Gravity Sewer System Operation and Maintenance

Operation and maintenance activities for a conventional gravity sewer system consist of cleaning sewers, monitoring sewers for illegal inflow connections, and pump station operation and maintenance. Pump station O&M involves repair and maintenance of mechanical equipment and electrical energy costs.

Table 6.6 shows the estimated O&M costs for the first year of operation. For the purposes of this study, it was assumed that two workers and a high-pressure cleaning truck would be required for sewer maintenance. The two-person sewer maintenance crew could clean all sewers at least once a year in addition to making repairs as necessary. A separate two-person crew would be required for pump station and force main maintenance. An annual allowance for materials for sewer and pump station maintenance is included in the total annual O&M cost estimate. Energy costs are based on sewage flow projections for the first year of operation and on a power cost of $0.08/kWh.

TABLE 6.6

CONVENTIONAL GRAVITY SEWER SYSTEM
ESTIMATED FIRST YEAR O&M COSTS

Item	Estimated Cost
Labor - 4 employees @ $30,000/yr	$120,000
Materials	25,000
Energy - Pump Stations 1 thru 6	20,000
TOTAL	$165,000

PRESSURE SEWER SYSTEM

Pressure Sewer System Description

A pressure sewer system can be considered to be the reverse of a water distribution system. In a pressure sewer system, sewage is pumped at its source into a sewer system which is maintained under pressure. Once in the sewer system, sewage flows to its destination under controlled hydraulic gradients. A pump is located at each residential or commercial unit or group of units to pump sewage into the collection system. Hydraulic gradients in the sewers are maintained by the individual pumping units and controlled by valves or standpipes on the sewers. Since pressure sewers flow full under a controlled hydraulic gradient, it is possible to use small-diameter pipe which can be buried at shallow depths conforming to the ground contour.

During periods of little or no flow, pressure sewers should remain full and under pressure. If not pressurized during low-flow periods, pressure sewers will drain and fill with air and may be subject to vacuum conditions. Large quantities of entrapped air may be difficult to remove from the system and would act to throttle sections of line thereby increasing head losses and decreasing hydraulic capacity of the system. Vacuum conditions in the lines could cause pipe collapse and may induce water column separation.

Pressure sewers which generally flow uphill naturally remain full and under pressure during low-flow periods due to static head. However, a pressure sewer which slopes downward will tend to drain during low-flow conditions. Downward sloping pressure sewer lines also may be subject to water column separation during abrupt reductions in flow such as during a neighborhood power outage. The momentum and weight of flow on a downward section of pipe will induce column separation and vacuum conditions in the line. Water column separation may cause severe hydraulic transients which could damage the system.

Standpipes or pressure-sustaining devices have been used to prevent downward sloping pressure sewers from draining and to maintain a minimum upstream hydraulic gradient while the system is in a low-flow period. Standpipes utilize static head by creating a high point in the system downstream of the sewer section which would otherwise drain. An alternative to a standpipe on a downward sloping section of the line would be to allow the section to act as a gravity flow line during low-flow periods. This could be accomplished with vacuum valves at the upstream location of the section subject to draining and air release valves at the downstream location of the section. Vacuum valves would be sized to prevent vacuum conditions in the line by allowing large quantities of air to be drawn into the line during the transition from pressure flow to gravity flow. When the pressure flow regime resumes, the downstream air release valves would vent the air from the system.

Air release valves are installed at all high points of pressure sewers to release accumulated air from the system. Automatic air release valves are provided at locations where air is expected to accumulate rapidly. Manual air release valves may be used where air release will be required infrequently. Performance of a pressure sewer system is dependent on proper release of air and gases as they accumulate at high points. Liberal placement of air release valves is therefore encouraged. Gate valves are provided at strategic locations in the system to isolate sections of lines or to bypass flow during an emergency. Cleanouts are provided for cleaning access. Pressure monitoring stations are provided at key locations in a pressure sewer system to assess performance and capacity of the system.

Individual pumping units provided at each sewage source to pump sewage into the pressure sewer system may be one of two types. The first type is known as a septic tank effluent pump (STEP) and the second is a grinder pump type. Besides major differences in on-lot facilities for STEP and for grinder pump systems, some subtle differences also exist in pressure main and treatment facilities design. On-lot facilities for each individual pumping unit consist of the pump, controls, piping, appurtenances and all other necessary equipment up to the connection with the pressure sewer main. To avoid the cost of a separate power supply system for the on-lot pumps, the power supply for each pump is obtained via the user's power connection. The user must therefore pay the electrical cost of running the on-lot pump which would be in the order of $5.00 per year. Since the on-lot facilities are owned by the entity providing sewage collection service, an easement must be granted by the user to the sewering entity to allow access to the on-lot facilities. STEP and grinder pump on-lot facilities are discussed in detail in the following sections.

STEP On-Lot Facilities

In a STEP system, sewage is first discharged to a septic tank for removal of solids and scum. Effluent from the septic tank is pumped into the pressure sewer system for transmission to the treatment facilities. Digested sludge and scum (septage) are removed from the septic tank as required at intervals similar to those of a typical septic tank/leach field system. It is estimated that septic tanks provide primary treatment to the following degree:

- 80 to 90 percent grease removal
- 60 to 90 percent suspended solids removal
- 50 to 80 percent BOD reduction.

Removal of the majority of the greases and solids from the sewage through the septic tanks simplifies pumping and reduces clogging and grease build-up in the small-diameter pressure sewers and at air release valves. The treatment afforded by the septic tanks reduces the strength of the sewage and thereby reduces treatment facility requirements.

The pump assembly is placed directly into the septic tank where it is suspended from an opening in the top of the tank. The assembly is contained in a vault with entry ports located between the sludge and scum layers in the tank to avoid intake of solids. A submersible well-type pump in the vault is controlled by mercury float level switches. Three switches are provided in the vault for pump on, pump off and high-water alarm. A mesh screen in the vault prevents intake of solids into the pump. A control panel is provided for each unit above grade where a high-level alarm is indicated by a red light. The user must notify maintenance personnel when the alarm light is on. In the event of pump failure or power failure, the septic tank provides emergency storage volume.

Discharge piping to the pressure sewer is provided with a check valve to prevent backflow from the sewer into the septic tank and an isolation gate valve. A check valve and an isolation valve are also provided as part of the pumping assembly in the pump vault. The check valve at the pump prevents the lateral line from draining back into the septic tank and filling with air which would be forced into the collection system during the next pump cycle. The pump check valve also provides a back-up to the lateral check valve. A typical STEP unit serving one or two homes will have a 1-1/4 inch diameter discharge line. Typical STEP on-site facilities are shown in Figure 6.2.

In areas with existing septic tanks there are two options available for STEP system installation. If the existing septic tanks are generally in good condition, a pump assembly can be placed in each existing tank. The second option is to abandon the existing tanks and install a single new interceptor tank and STEP assembly for every house or for two or three houses. If existing tanks are in poor condition, they may not be watertight and could be subject to infiltration or exfiltration, thus justifying abandonment. New development would have the option of using a single septic tank and STEP assembly to serve one, two or three users.

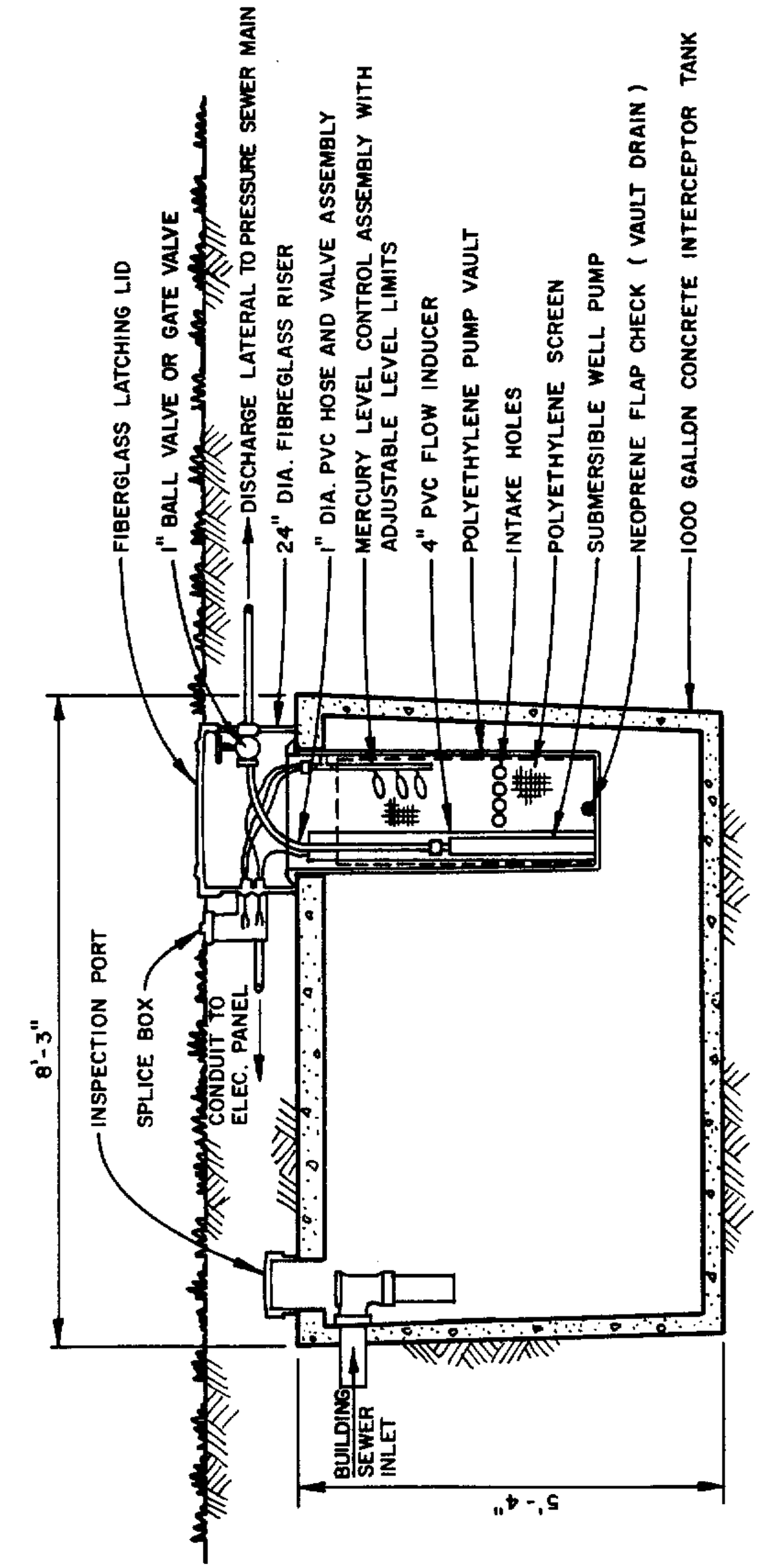
When a single STEP unit serves two or more users, one user must provide the power connection, thus necessitating the need for power cost-sharing arrangements. Larger capacity pumps are available for large flows from septic tanks serving commercial users, apartment complexes or community systems.

<u>Grinder Pump On-Lot Facilities</u>

Grinder pump systems are different from STEP systems in that solids in the sewage are not removed but are macerated into a slurry as the sewage is pumped into the pressure sewer system. Septic tank operation is therefore not required. Grinding solids into a slurry allows the use of small-diameter pipe.

Sewage flows from its source directly into a wet well containing a grinder pump assembly. Control and arrangement of a grinder pump assembly are similar to the STEP assembly. Float level switches provide pump on, pump off and high-level alarm signals to a control panel mounted above grade. A check valve is provided at the connection to the pressure main and also on the pump discharge pipe in the wet well. A gate valve is provided near the pump unit and a corporation stop is located at the main line connection. A typical grinder pump unit serving one or two homes will have a 1-1/4 inch diameter discharge line. Due to the

FIGURE 6.2



TYPICAL STEP ON-LOT FACILITIES

power requirements associated with grinding solids, the grinder pump minimum motor size is two horsepower compared with 1/3 horsepower for a typical STEP pump. A typical grinder pump installation is shown in Figure 6.3.

As with STEP systems, the grinder pump unit may service one or more users. Unlike the STEP system, however, the grinder pump wet well provides little storage during pump failure or power failure. If a grinder pump system is used, existing septic tanks could possibly be used for overflow storage, but would likely be abandoned.

Pressure Sewer System Technical Feasibility

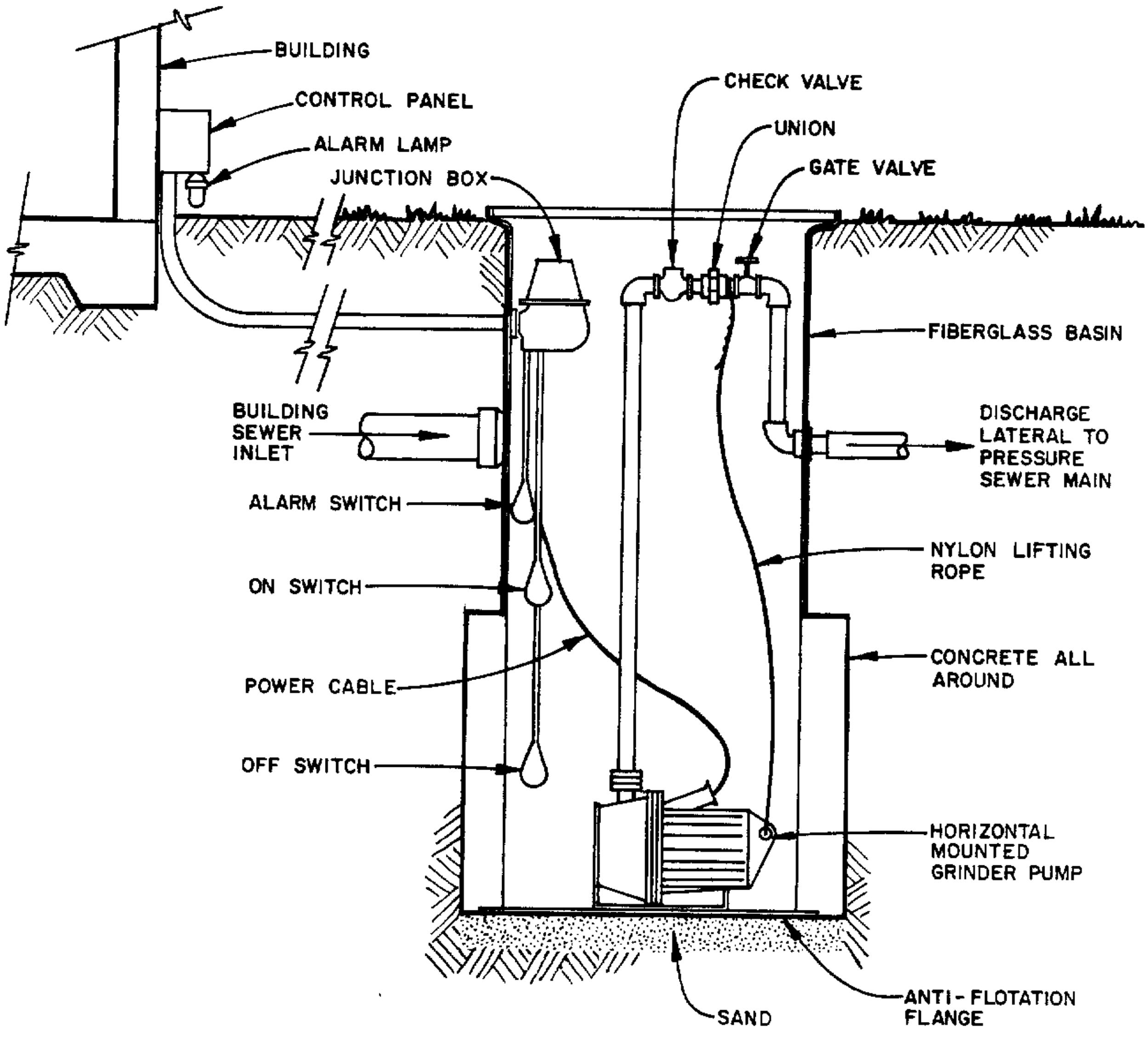
A list of existing pressure sewer systems nationwide is provided in Appendix F. Pressure sewer systems have proven to be a viable alternative to conventional gravity sewers at the locations listed in Appendix F. The STEP system which services the community of Glide, Oregon is one of the oldest and largest pressure sewer systems. The Glide system has been fully operational since March 1980 and has made substantial contributions to pressure sewer technology, especially for STEP systems (Reference 6.6). Pressure sewer technology has reached the point where most of the "bugs" have been discovered and remedied by design changes.

The systems at Glide and elsewhere have shown that pressure sewers are technically feasible and economical under certain circumstances. However, existing pressure sewer installations have also shown that careful planning and design are necessary to insure that systems are not plagued by long-term maintenance problems.

Pressure Sewer System Advantages

Since pressure sewer systems function independently of gravity, they provide a feasible alternative to conventional gravity sewers in areas with difficult terrain, sparse population, or other conditions which make gravity sewers impractical and uneconomical. Pipeline construction of small-diameter pressure sewers buried at shallow depths is less costly than conventional sewers, especially in areas with high groundwater or shallow bedrock. This cost advantage is enhanced if a number of pumping stations would be required with a conventional system. However, much of the pressure sewer pipeline cost savings may be offset

TYPICAL GRINDER PUMP ON-LOT FACILITIES



REFERENCE 6.8

by additional capital and O&M costs associated with mechanical equipment such as pumps and valves. If conditions are such that the costs of a conventional system will be exceedingly high, a pressure sewer system offers the versatility to overcome some adverse conditions and be used in place of a conventional system at a lesser cost. If conditions are suitable for a conventional sewer system, however, the cost of a pressure system will likely be higher or comparable to a conventional system.

Therefore, the primary advantage of the pressure sewer system has historically been that it can be used in areas where the application of a conventional gravity sewer system is either not feasible or too costly due to difficult conditions. In such difficult conditions a pressure sewer system can be constructed at significant cost savings over a conventional gravity sewer system. The area served by the Glide project, for example, is sparsely populated and has terrain unsuitable for gravity sewers. If a gravity sewer system had been constructed at Glide, 19 mainline pump stations would have been required and 48 on-lot facilities would still have been necessary to sewer a population of approximately 2,000 (Reference 6.6).

An additional cost advantage of pressure sewer systems is that costs are deferred, thus taking advantage of the time value of money. One way that costs are deferred in a pressure system is by construction of a system that may be less expensive to build, but more expensive to maintain than a conventional system. A significant amount of pressure system capital cost is also deferred in the form of on-lot facilities for future development. The sewer system is constructed to accommodate future development, but the on-lot facilities are not put in place until development is underway. Developers could be given the responsibility to provide the on-lot facilities to County specifications as a condition of connection to the sewer system in addition to a connection charge for the sewer system and treatment facilities.

Another advantage of pressure systems is that there is little or no I/I in the system. Infiltration may be introduced to the system only at septic tanks or wet wells which are not watertight. Inflow from down-outs or sump pumps could enter the system only through connections to building plumbing. Elimination of I/I reduces flow requirements in the collection system and at the treatment facilities. Reduced flow at the treatment facilities results in reduced capital and O&M costs.

STEP System Advantages

Removal of solids and scum from sewage in a STEP system simplifes pumping. Efficient, reliable pump types may be used in place of typical sewage pumps. Clogging of pumps, pressure sewers or air release valves is avoided since solids are removed from the flow prior to pumping. It is expected that cleaning of STEP pressure sewers will be needed much less frequently than grinder pump pressure sewers. During pump failure or power failure the septic tank provides emergency storage of sewage. A STEP system provides partial sewage treatment in the on-lot septic tanks which act as clarifiers and long-term anaerobic sludge digesters.

Grinder Pump System Advantages

Because grinder pump systems do not rely on septic tanks to remove solids from the sewage, negative aspects associated wtih septic tanks are avoided. The capital cost of a grinder pump wet well will be less than that of a STEP septic tank. Septicity problems of septic tank effluent discussed in the following sections are avoided because sewage is pumped into the pressure sewer system shortly after it enters the grinder pump wet well. The potential for odor, corrosion and treatment facility impacts associated with a grinder pump system is much less than with a STEP system. In fact, grinding of sewage solids at the source precludes the necessity for solids grinding at the treatment facilities. Grinder pump units thereby provide a primary treatment process. Also, pumping of septic tank septage is not necessary with grinder pumps.

Pressure Sewer System Disadvantages

The primary disadvantage of pressure sewer systems is the level of O&M effort required. The proliferation of mechanical equipment such as pumps and valves increases the potential for failure and necessitates a rigorous preventative maintenance and monitoring program. Pressurized sewers also have the potential to burst or leak, resulting in spillage of partially treated sewage in the case of a STEP system or untreated sewage in grinder pump systems. The high cost of O&M for a pressure sewer system in the long term may significantly offset capital cost savings over a conventional gravity sewer system.

The successful application of pressure sewer systems has hinged on the inability of conventional sewers to overcome economically adverse conditions. However, if conditions are conducive to conventional sewers, pressure sewers may have a higher capital cost in addition to higher O&M costs.

STEP System Disadvantages

Sewage in a septic tank is maintained in an anaerobic or septic state in which nuisance gases such as hydrogen sulfide, methane, carbon dioxide and carbon monoxide are produced. Hydrogen sulfide is highly corrosive to many materials and has a strong, offensive odor. Materials for on-lot pumping facilities and pressure sewer valves and standpipes must be carefully selected to be inert to hydrogen sulfide. Air release valves often must discharge to soil absorption beds to prevent an odor nuisance caused by venting trapped gases. Treatment facilities must incorporate hydrogen sulfide and odor control systems.

Another disadvantage of STEP systems is operation of on-lot septic tank facilities. Accumulated digested sludge and scum must be removed from the tank and disposed of as required. However, removal intervals are expected to be long. Equipment to remove and dispose of septage is a necessary component of a STEP system. The treatment facilities must be designed to be capable of receiving and processing septage.

Grinder Pump System Disadvantages

Due to the harsh operational requirements of the grinder pump, more pump failures and clogging and shorter pump life are anticipated than for STEP pump units. Grinder pumps must have larger horsepower motors than STEP pumps to have adequate power to be able to macerate solids. Larger motor requirements mean higher pump cost and higher power costs. Grease buildup on the interior of pressure sewers and on air release valve orifices occurs with grinder pump systems. Clogging of the sewer lines and valves is more likely with grinder pumps than with STEP pumps and cleaning of grinder pump sewer lines and valves must be done more frequently than with STEP systems. Clogged sewer lines will increase head losses in the system and clogged air release valves may allow accumulation of air in the system which would further impair the hydraulic capacity of the lines. In the event of pump failure or power failure, the storage capacity of the grinder pump wet well is limited. Because of the solids in a grinder pump pressure sewer system, scouring velocities in the sewers must be high, thereby increasing friction losses, pumping costs and pressure in the system.

Pressure Sewer System Conceptual Design

A preliminary layout of a pressure sewer system to serve the Los Osos area has been developed and is shown in Figure 6.4. Design criteria for the pressure sewer system are given in Table 6.7. Although there may be subtle differences between a STEP pressure sewer system and a grinder pump sewer system, the layout and sizing of the pressure sewers will be the same for either type of system. The layout of the collection system generally corresponds to the layout of the gravity sewer system since both systems must serve the same areas. In order to avoid high discharge heads for the on-lot facilities and to keep the pressure in the system at an acceptable level, the system is divided into two pressure zones. In general, the lower presure zone is the area between the shoreline and the 60-foot contour, while the upper pressure includes the area above the 60-foot contour.

Two pump stations are required to pump flow from the lower pressure zone into the upper pressure zone. The location of the two pump stations and their tributary areas is shown on Figure 6.4. The design criteria for the pump stations are listed in Table 6.8. Pumping capacities of the pump stations were determined based on the number of existing lots and the potential for future development with the tributary area of each station. The pump stations are located above the 60-foot contour so that the lower pressure zone system is maintained under constant pressure. At downward sloping pressure sewer sections in the upper pressure zone which may be subject to de-pressurization, vacuum breaker valves and air release valves will be used to avoid water column separation in lieu of standpipes.

PRESSURE SEWER SYSTEM PRELIMINARY LAYOUT

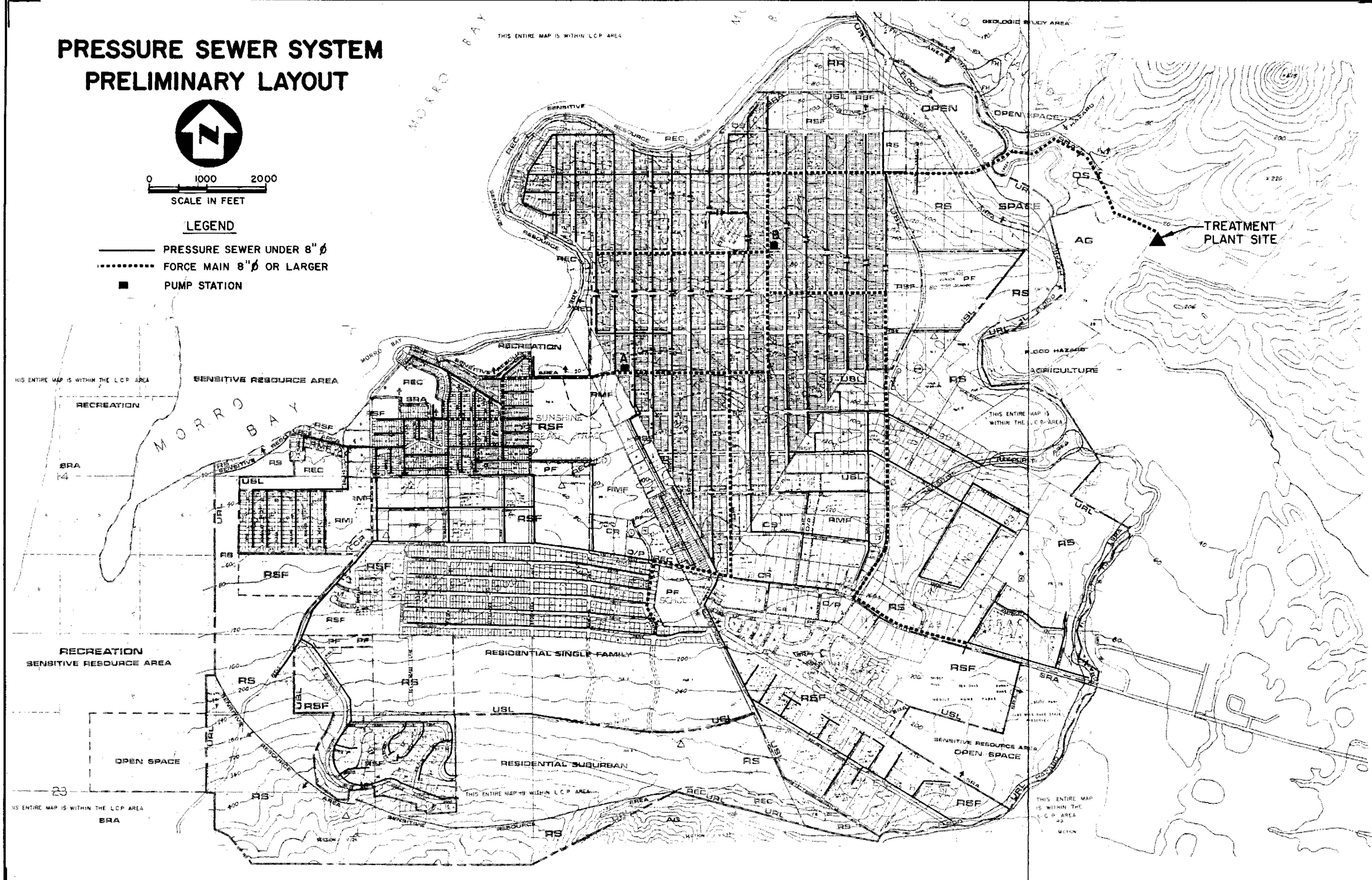


TABLE 6.7

PRESSURE SEWER SYSTEM - PRELIMINARY DESIGN CRITERIA

Item	Criteria
Pressure Sewers	
Minimum pipe size	3-inch diameter
Maximum system pressure	60 psi
Pipe material	PVC
Minimum depth of cover	3 feet
Air release valves	Provide at high points
Blow-down assemblies	Provide at low points
On-Lot Facilities	
Type	STEP
Pump type	Submersible vertical turbine
Typical pump horsepower	1/3
Typical discharge lateral size	1-1/4 inch diameter
Discharge lateral pipe material	PVC
Alarm actuation	High tank level

TABLE 6.8

PRESSURE SEWER SYSTEM
PUMP STATION PRELIMINARY DESIGN CRITERIA

Pump Station	Design Capacity mgd	Design Capacity gpm	Design TDH (ft)	Total HP Reqd.	Standby Power	Pump Type	Pump Speed Control	Odor Control Equip.
A	0.7	490	110	20	permanent	vert. turbine	variable	yes
B	0.5	350	75	10	permanent	vert. turbine	variable	yes

[a]Assuming 70 percent overall pump efficiency.

Pressure sewer preliminary sizes were selected based on flow criteria adopted from a Batelle Institute report concerning pressure sewer design considerations (Reference 6.9). Actual design of the pressure sewer system will be based on flows developed through computer simulation of the probability of a certain number of pumps discharging to a pressure sewer line simultaneously. Computer network analysis would also be necessary to select optimum pipe sizes and layout and to determine discharge head requirements for the individual pumping units.

Existing gravity sewers and cluster septic tank/leach field systems serving the mobile home parks and the Bayridge Estates and Vista de Oro subdivisions will require separate, large pump units. These large units are considered separately as cluster pump stations.

An outfall-type interceptor will transport sewage from the service area to the treatment facilities as with the conventional gravity sewer system. Due to the lack of I/I in the pressure sewer system, the interceptor size is smaller than for the conventional gravity sewer system. As with the conventional system, the location of pump stations, the alignment of the pressure sewer interceptor pipelines, pump station capacities and pressure sewer sizes will likely be modified during the design phase of the project. However, for the comparison purposes of this study, the conceptual design shown in Figure 6.4 will be adequate.

<u>On-Lot Facilities</u>

The following alternatives were initially considered for on-lot facilities:

- One STEP unit including new interceptor tank at each connection
- One grinder pump unit at each connection
- A STEP pump unit in each existing septic tank with an interceptor tank and STEP unit for each new connection
- One STEP unit with new interceptor tank to serve an average of two connections
- One grinder pump unit including wet well to serve an average of two connections

Cost data indicate that the capital cost of a grinder pump on-lot unit is comparable to that of a STEP on-lot unit. However, energy, maintenance, and replacement costs are generally higher for grinder pump systems as discussed in the preceding sections. Most existing pressure sewer systems utilize STEP units to avoid the disadvantages of pumping solids and grease material through the pressure sewer system. STEP units are therefore recommended as the on-lot facilities for consideration of a pressure sewer system in Los Osos and will be evaluated exclusively throughout the remainder of this chapter.

Due to the uncertainties of infiltration and exfiltration associated with the use of existing septic tanks as STEP interceptor tanks, this conceptual design does not consider use of existing tanks. However, if the pressure sewer system alternative is implemented, the feasibility of using at least some of the existing septic tanks should be investigated.

The following alternatives were therefore selected from the initial considerations for detailed cost evaluation:

<u>Alternative A</u>: One STEP unit including new interceptor tank at each connection.

<u>Alternative B</u>: One STEP unit including new interceptor tank serving two connections.

<u>Alternative C</u>: One STEP unit including new interceptor tank serving two connections, located in County right-of-way and provided with a separate power source and traffic loading access covers.

Alternatives A and B present typical pressure sewer system arrangements. In Alternative A, each user has its own STEP unit, located on that user's lot and provided power service from that user's power connection. Alternative B proposes to use one STEP unit to serve two users in order to reduce the STEP unit capital costs, which represent a significant portion of the total pressure sewer system capital cost. However, the STEP unit must be placed on one of the two lots it serves and must be provided a power connection from that lot. Alternative B has the potential to create many difficulties in administering placement of the STEP units.

Alternative C was considered to circumvent foreseeable problems of Alternative B while preserving the cost savings of one STEP unit per two users. In order to avoid unevenly placing the burden of locating and providing power to shared STEP units on some users and not others, Alternative C proposes to have one STEP unit serve two users but to place the STEP units in the County right-of-way with a separate independent power source. The STEP unit access risers and covers which are normally provided are not capable of withstanding traffic loadings. Therefore, the STEP units considered for Alternative C will have an additional cost associated with providing traffic loading access risers and covers necessary to place the units in the County right-of-way.

Future development within the Los Osos service area for all pressure sewer alternatives would connect directly to the pressure sewer main so no service laterals would need to be provided for future connections. However, new on-lot facilities would be added to the system every year to service new development. Capital costs for future on-lot facilities are not included in the initial construction capital cost estimate but are considered as future capital expenditures in the present worth analysis section of this chapter.

Pressure Sewer System Unit Costs

Unit costs for pressure sewers include costs for the following items:

- Pavement saw cutting
- Trench excavation
- Pipe bedding material and placement
- Pipe material and installation
- Pipe zone backfill material, placement and compaction
- Backfill and compaction of remainder of trench with native material
- Repair of water and gas service connections
- Pavement repair and replacement
- Air release valves with odor treatment features, air vacuum valves and gate valves
- Cleanouts
- Testing and cleanup

Valves and other appurtenances on the pressure sewer system are not considered separately for the conceptual design cost estimate. The actual number and type of appurtenances will be a function of subsequent design efforts. Costs for these appurtenances are incorporated into the unit costs for pressure sewers as discussed in the following section. Design criteria for the pressure sewer system, including typical STEP pumping units, are listed in Table 6.7.

Unit cost for the on-lot facilities is based on a STEP system and includes material and installation costs for the STEP interceptor tank/pump assembly, discharge lateral, connection to the sewer main, controls and control panel, and power connection. The STEP assembly and discharge lateral include all appurtenances shown in Figure 6.2. Connection to the interceptor tank and abandonment of existing septic tanks are not included. A unit cost of $2,000 was used for materials and installation for the complete STEP unit based on recent bid prices for pressure sewer systems (Reference 6.10).

The cost for the power supply system in Alternative C was developed based on providing individual power systems for each block of STEP units. Each individual block system would include a connection to the primary power source, a meter and main switchboard, conduits and cable in the right-of-way, conduits and cable up to the connection with each STEP control panel, and pull boxes. A cost estimate for such a power supply system was developed for a typical block. In addition to the above, costs include trenching, backfilling, compaction, and concrete encasement for conduits. A per STEP unit cost was calculated from the typical block cost estimate and applied to the total number of STEP units. Unit cost for on-lot unit traffic covers in Alternative C was based on recent construction costs for such covers.

Costs for pressure sewer pump stations were developed and were found to be comparable to conventional gravity sewer pump stations. Costs for the pump stations were therefore determined based on their performance requirements as compared with the performance requirements and costs of conventional gravity sewer pump stations. Measures to mitigate odor and corrosion effects of effluent septicity and standby power facilities are included for the pressure sewer pump stations.

Standby power is provided in the event that some or all of the on-lot facilities would not be affected by an outage affecting the the pump stations. A summary of the conceptual design components of the three pressure sewer alternatives and their respective cost estimates are given in Tables 6.9, 6.10, and 6.11.

Pressure Sewer System Operation and Maintenance

Operation and maintenance requirements for a pressure sewer system include the following (Reference 6.11):

- Cleaning and maintenance of air release valves
- Actuation of manual air release valves
- Gate valve exercising
- Cleaning or pigging of pressure sewer lines
- Repair of damaged pressure sewers
- Pressure monitoring
- On-lot or R/W pump unit repair or replacement
- Pump station maintenance
- On-lot or R/w pump unit power costs
- Pump station power costs
- Septic tank pumping costs

Table 6.12 shows the estimated O&M costs for the first year of operation. It is assumed that six maintenance personnel will be required to operate and maintain the proposed pressure sewer system. A two-person crew with a pumper truck will be assigned to pump septage from septic tanks on a full-time basis. Assuming this crew can pump two tanks per working day, the pumping frequency for one unit serving two users would be approximately 5 years and would be approximately 10 years for one unit serving one user. The pumper truck would also be available to empty tanks whose pump unit had failed. A second two-person crew will be responsible for maintenance of the pressure sewer system. The third two-person crew will operate and maintain the two main pump stations and the on-lot pump units.

TABLE 6.9

PRESSURE SEWER SYSTEM - PRELIMINARY CAPITAL COST ESTIMATE
ON-LOT FACILITIES ALTERNATIVE A - ONE UNIT PER USER

Item	Quantity	Unit Cost ($)	Total Cost ($)
Pressure Sewers			
3-inch diameter	184,000 lf	16.00/lf	2,944,000
4-inch diameter	17,000 lf	17.00/lf	289,000
6-inch diameter	9,000 lf	21.00/lf	189,000
8-inch diameter	15,000 lf	23.00/lf	345,000
10-inch diameter	6,000 lf	28.00/lf	168,000
16-inch diameter	7,000 lf	48.00/lf	336,000
Pump Station A	--	--	250,000
Pump Station B	--	--	200,000
Cluster System Pump Stations	7	50,000 ea.	350,000
On-Lot Pump Facilities	4,200[a]	2,000 ea.	8,400,000
Gravity Interceptor Sewer			
18-inch diameter	8,300 lf	55.00/lf	457,000
SUBTOTAL			13,928,000
Contingency (20%)			2,786,000
SUBTOTAL			16,714,000
Contractor's Overhead and Profit (15%)			2,507,000
SUBTOTAL (Construction Costs)			19,221,000
Technical Services			
Basic Design Services (5.9%)			1,134,000
Right-of-Way Easement Acquisition (4%)			769,000
Other Technical Services (12%)[b]			2,307,000
TOTAL			23,431,000

[a] Value cannot be directly correlated with Table 2.1 because certain assumptions were made regarding number of multiple units per connection to account for apartments, etc.
[b] Includes geotechnical, surveying, construction management, engineering, legal, financial and administrative services.

NOTE: ENR CCI 5180

TABLE 6.10

PRESSURE SEWER SYSTEM - PRELIMINARY CAPITAL COST ESTIMATE
ON-LOT FACILITIES ALTERNATIVE B - ONE UNIT PER TWO USERS

Item	Quantity	Unit Cost ($)	Total Cost ($)
Pressure Sewers			
3-inch diameter	184,000 lf	16.00/lf	2,944,000
4-inch diameter	17,000 lf	17.00/lf	289,000
6-inch diameter	9,000 lf	21.00/lf	189,000
8-inch diameter	15,000 lf	23.00/lf	345,000
10-inch diameter	6,000 lf	28.00/lf	168,000
16-inch diameter	7,000 lf	48.00/lf	336,000
Pump Station A	--	--	250,000
Pump Station B	--	--	200,000
Cluster System Pump Stations	7	50,000 ea.	350,000
On-Lot Pump Facilities	2,100[a]	2,000 ea.	4,200,000
Gravity Interceptor Sewer			
18-inch diameter	8,300 lf	55.00/lf	457,000
SUBTOTAL			9,728,000
Contingency (20%)			1,946,000
SUBTOTAL			11,674,000
Contractor's Overhead and Profit (15%)			1,751,000
SUBTOTAL (Construction Costs)			13,425,000
Technical Services			
Basic Design Services (5.9%)			792,000
Right-of-Way Easement Acquisition (4%)			537,000
Other Technical Services (12%)[b]			1,611,000
TOTAL			16,365,000

[a] Value cannot be directly correlated with Table 2.1 because certain assumptions were made regarding number of multiple units per connection to account for apartments, etc.

[b] Includes geotechnical, surveying, construction management, engineering, legal, financial and administrative services.

NOTE: ENR CCI 5180

TABLE 6.11

PRESSURE SEWER SYSTEM - PRELIMINARY CAPITAL COST ESTIMATE
ON-LOT FACILITIES ALTERNATIVE C - ONE UNIT IN R/W PER TWO USERS

Item	Quantity	Unit Cost ($)	Total Cost ($)
Pressure Sewers			
3-inch diameter	184,000 lf	16.00/lf	2,944,000
4-inch diameter	17,000 lf	17.00/lf	289,000
6-inch diameter	9,000 lf	21.00/lf	189,000
8-inch diameter	15,000 lf	23.00/lf	345,000
10-inch diameter	6,000 lf	28.00/lf	168,000
16-inch diameter	7,000 lf	48.00/lf	336,000
Pump Station A	--	--	250,000
Pump Station B	--	--	200,000
Cluster System Pump Stations	7	50,000 ea.	350,000
R/W Pump Facilities	2,100[a]	2,000 ea.	4,200,000
R/W Pump Power Supply System	--	--	3,300,000
R/W Pump Traffic Weight Access Covers	2,100[a]	500 ea.	1,050,000
Gravity Interceptor Sewer			
18-inch diameter	8,300 lf	55.00/lf	457,000
SUBTOTAL			14,078,000
Contingency (20%)			2,816,000
SUBTOTAL			16,894,000
Contractor's Overhead and Profit (15%)			2,534,000
SUBTOTAL (Construction Costs)			19,428,000
Technical Services			
Basic Design Services (5.9%)			1,146,000
Right-of-Way Easement Acquisition (4%)			777,000
Other Technical Services (12%)[b]			2,330,000
TOTAL			23,670,000

[a]Value cannot be directly correlated with Table 2.1 because certain assumptions were made regarding number of multiple units per connection to account for apartments, etc.

[b]Includes geotechnical, surveying, construction management, engineering, legal, financial and administrative services.

NOTE: ENR CCI 5180

TABLE 6.12

PRESSURE SEWER SYSTEM
ESTIMATED FIRST YEAR O&M COSTS

Item	Estimated Cost
Labor - 6 employees at $30,000/year[a]	$180,000[a]
Materials	25,000
Energy	
STEP units	12,000
Pump Stations A and B	4,000
TOTAL	$271,000[a]

[a]For Pressure Sewer Alternative A, 7 employees were assumed. Estimated labor is $210,000 and the Total Estimated Cost is $301,000.

With regard to the two on-lot and one R/W facilities alternatives, it was assumed that energy O&M costs would remain the same for each alternative. Although Alternative A proposes twice as many pumps as Alternatives B and C, the pumps in Alternative A will only operate half the time of the other pumps. On the other hand, additional labor (assumed as one more person) is required for Alternative A, because it contains twice as many pump units as Alternatives B and C.

An annual allowance for materials costs is included in the pressure sewer system O&M costs. Not included in the materials allowance are replacement costs for failed on-lot or R/W pump units because they will be under warranty. On-lot and R/W unit replacement costs are considered in the present worth analysis of this chapter. Energy costs were calculated based on sewage flow projections for the first year of operation and a power cost of $0.08/kWh. Energy costs for on-lot pump units are based on STEP units. Grinder pump powers costs would be higher than STEP pumps.

VARIABLE-GRADE GRAVITY SEWER SYSTEM

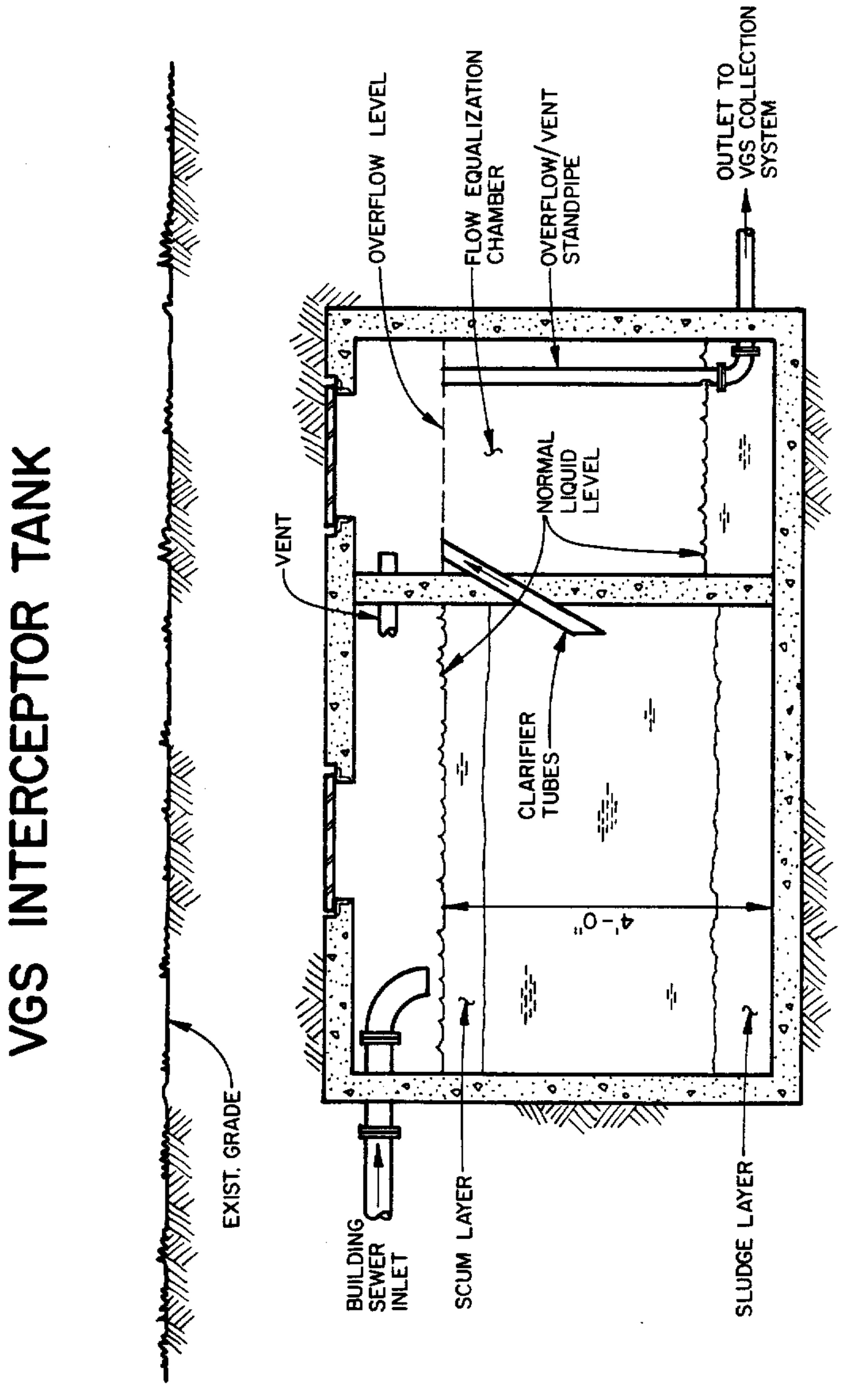
Variable-Grade Gravity Sewer System Description

The variable-grade gravity sewer (VGS) concept is a modification of the STEP system. Effluent from individual septic tanks is conveyed through small-diameter pipes buried at shallow depths following the ground contours. Instead of pumping effluent from the septic tanks, the VGS system utilizes differential static head between the tank outlet and the system discharge point to drain the effluent to the discharge point. The discharge point typically would be a treatment facility or pump station.

So long as each septic tank outlet on the system is higher than the hydraulic gradient of the sewer, effluent will drain from the septic tank to the discharge point. Each low point in the sewer will constantly be full of standing water but will drain in the same manner as a sink drain trap. The hydraulic gradient of the sewer will follow the profile of the sewer except at low points where the hydraulic gradient will correspond with the water level at each side of the low point.

Since most of the solids are removed at the septic tank, solids settling and deposition at sewer low points should not occur. Removal of solids from the sewage also allows the use of smaller pipes than those required for conventional sewers where blockage is a consideration. As with the STEP system, digested sludge and scum which accumulate in individual septic tanks must be removed and disposed of at intervals comparable to a typical septic tank and leach field system.

In order to realize the advantage of minimum sewer pipe sizes, it is important to maintain a relatively constant flow rate from each individual septic tank. This flow equalization is accomplished with a specially designed septic tank which has a separate surge storage chamber or flow equalization chamber provided on the discharge end of the tank. When a surge of sewage is discharged into the tank, an equal volume of effluent is discharged from the tank to the surge storage chamber. Effluent is allowed to drain slowly from the chamber to the sewer main through an orifice outlet. A standpipe is used to vent the orifice outlet and to provide an overflow outlet. The recommended configuration for a surge storage chamber is shown in Figure 6.5.



REFERENCE 6.13

If existing septic tanks are to be used in a VGS system, it is recommended that a surge storage chamber be installed on the discharge end of the tank. A single surge storage chamber could possible be connected to two or more existing septic tanks. Other methods may be devised to equalize flow from existing septic tanks which would utilize the volume of the tank itself for surge storage. Foregoing flow equalization altogether would result in larger sewer lines for sewers serving only a small number of homes. Sewers draining a large number of homes may not be affected as much since peak flows would tend to be lower.

The surge storage chamber also provides for storage of effluent for septic tanks which are usually above the sewer hydraulic gradient, but which are temporarily below the hydraulic gradient during periods of high flow. When the peak flow condition is over, the hydraulic gradient lowers and effluent drains from the surge storage chamber to the sewer. The sewer system can be designed to avoid or minimize the number of septic tanks subject to elevated hydraulic gradients by lowering the downstream profile of the sewer and therefore lowering its hydraulic gradient. Elevated hydraulic gradients create the potential for backflow into individual septic tanks.

If a septic tank outlet is below the sewer or if a septic tank is located at a sewer low point containing standing water which will constantly be below the sewer hydraulic gradient, effluent must be pumped from the tank into the sewer. This pumping is different from the STEP system because pressure is not maintained in the sewer. Effluent is pumped to a surge storage chamber located above the sewer hydraulic gradient where it drains by gravity into the sewer. This arrangement provides flow equalization and prevents temporary pressurization of the sewer. Pressure in the sewer would elevate the sewer hydraulic gradient, potentially to a level which would cause backflow into individual septic tanks. If location of a surge storage chamber above the sewer hydraulic gradient is not possible, direct pumping into the sewer would be feasible but may require a larger sewer or additional pumping units.

As with conventional gravity sewers, pump stations and force mains are required with the VGS system as topography dictates. These pump stations would be similar to those used with conventional gravity sewers except that consideration of the differences between raw sewage and septic tank effluent would be necessary.

Those septic tank connections subject to backflow caused by temporary hydraulic gradient increases must be provided with backwater valves. Backwater valves are used in lieu of check valves because they stay open when not in use and provide a vent for the sewer line. Check valves must be provided on the discharge line of each pump unit to prevent backflow.

Vents are required on the sewer line as on household plumbing to maintain atmospheric pressure in the system to avoid backpressure and siphoning slugs. Vacuum conditions caused by inverted siphons can be avoided by providing a vent at all high points on the sewer. Although solids accumulation in the sewers is not expected, clean-outs should be provided for access to the sewer for cleaning and testing. Vents which can also function as clean-outs will reduce the number of clean-outs required.

Variable-Grade Gravity Sewer System Technical Feasibility

The VGS concept is quite new. Almost all the information available on the VGS system is based on an experimental pilot project in the community of Mt. Andrew, Alabama. The Mt. Andrew project involved 31 houses served by three separate sewer lines, two 3-inch and one 2-inch. The results of the project were published in May 1984 after the system had been in operation for about four years and seem to indicate that VGS systems are technically feasible (Reference 6.13). There are several VGS projects currently under way in California (Reference 6.14). Three communities in Ohio are also proceeding with VGS projects based on the Mt. Andrew study (Reference 6.15). All of the projects are in the planning or construction phases and in general are not expected to be operational until 1986 or 1987. A list of VGS projects is provided in Appendix G. Note that the communities which are undertaking VGS projects have much smaller populations than CSA No. 9.

The capacities of all three of the Mt. Andrew sewer lines were much greater than the effluent flow from the 31 houses. After 18 months of operation, the 2-inch sewer was unearthed at several locations and inspected. The inspected sections displayed no signs of solids accumulation despite concentrations of suspended solids in the effluent which were greater than that of typical septic tank effluent. While the information gained from these inspections is positive, it cannot be considered conclusive considering the short period of service and the low flow rates (Reference 6.13).

Many modifications to the VGS system were developed based on the four years of operation of the Mt. Andrew system. Some of these modifications have yet to be tested. The surge storage chamber was an idea spawned from the Mt. Andrew project. However, this concept has not been tested and clogging of the outlet orifice (3/16-inch diameter is recommended) may be a significant operational problem.

The VGS projects now under way are expected to provide the answers to many of the remaining questions regarding VGS systems. While the Mt. Andrew system proved that the VGS concept will work on a small scale, it has yet to be conclusively proven on a larger scale. It is apparent that some of the VGS technology must undergo further refinement in addition to that provided by the operational experience at Mt. Andrew. However, the Mt. Andrew system did indicate that VGS is technically feasible and will work under the right conditions.

Variable-Grade Gravity Sewer System Advantages

The primary advantages of the VGS system are low construction cost and low O&M cost. Construction of a VGS system involves relatively small pipe buried at shallow depths. Removal of solids from the flow and equalization of peak flows allow the use of small-diameter pipe. Removal of solids from the flow allows the profile of the sewer to follow the ground contours because solids deposition will not occur. Manholes are also dispensed with because of the lack of solids in the flow. The small pipe size reduces material and handling costs and the shallow depth reduces excavation and groundwater dewatering costs. Sewer grade and alignment are ideally not critical for VGS which makes excavation and pipe laying procedures simpler and less costly. Since individual pumping units are ideally not required, material, installation, capital and O&M costs for these units are avoided. The difference between the cost of clean-outs and vents on VGS and the cost of manholes for conventional sewers represents substantial savings for a VGS system.

Sewer maintenance requirements are comparable to conventional gravity sewers and will probably be less due to the fact that solids are not transported in the VGS system. O&M costs for pump stations and force mains would also be comparable to similar conventional gravity sewer facilities, although effluent septicity may create additional maintenance requirements.

Variable-Grade Gravity Sewer System Disadvantages

Disadvantages of the VGS system which can be anticipated are similar to STEP system disadvantages relating to the use of septic tanks to remove solids from the sewage. These disadvantages include the effects of effluent septicity resulting in corrosion, odor, and potential treatment problems. The effects of effluent septicity can be mitigated through the use of corrosion-resistant materials, odor control facilities and by designing treatment facilities to accommodate septic tank effluent. However, the vented sewer lines create the likelihood for odor problems throughout the service area.

Accumulated digested sludge and scum must be removed from individual septic tanks at intervals similar to a typical septic tank and leach field system. Equipment and facilities to remove and dispose of septage are necessary components of a VGS system. The treatment facilities must be designed to be capable of receiving and processing septage. Direct landfill disposal of septage would be an alternative to treatment plant processing since, in effect, the sludge will have already been treated.

Another disadvantage of the VGS system is that, even more so than conventional sewers, its successful application is dependent on the topography of the area to be sewered. If the topography of the area is not ideally suited to the VGS system, its advantages over conventional gravity sewers become less apparent as sewer size and depth increase. If many septic tanks require pumps, the capital and O&M cost advantages over pressure sewers also diminish. The VGS system definitely does not have the flexibility of application that pressure sewers have, due to critical topographical constraints.

As noted above, the communities planning VGS projects are quite small, even compared to the Los Osos area. The application of VGS sewers appears to be restricted to very small communities for the following reasons:

- As the flow volume increases with increased population, flow rate will determine sewer diameter, not solids in the sewage.
- As the flow volume increases with increased population, peak factors will decrease naturally, thus negating the positive effects of VGS flow equalization.

An important negative consideration of the VGS system is the risk involved with investing in a system which has not been proven on a scale comparable to the CSA No. 9 service area, even though it apparently is technically feasible. The Mt. Andrew project may not have been extensive enough to adequately assess the disadvantages of the VGS system. Some disadvantages may be revealed only through long-term operation of an actual installation.

Variable-Grade Gravity Sewer Conceptual Design

A preliminary layout of a VGS system to serve the Los Osos area has been prepared. For this conceptual level design, the layout of the VGS system was assumed to correspond to the layout of the conventional gravity sewer system. Therefore, there are five pump stations, each located near the low point of the five drainage basins which do not drain directly to the treatment facilities and force main interceptors.

A new septic tank with flow equalization chamber would be provided for each existing user. Future users would be provided with VGS tanks as development occurred. Without detailed topographical data and a review of the hydraulic profile of each sewer, it is difficult to estimate accurately the number of on-lot pumping units required with a VGS system for the service area. Based on a review of available topographical data and the preliminary layout of the collection system, a rough estimate of 1,500 houses will require on-lot STEP pump units including flow equalization chambers. The design of the pump stations would be similar to the conventional gravity sewer pump stations, except that the design flows will be reduced due to the flow equalization and decreased I/I. Facilities for the control of hydrogen sulfide will be an additional requirement of VGS pump stations.

During the preparation of the VGS system conceptual design, it was determined that a VGS system would simply not work in much of the service area due to topographical constraints. Many users would require on-lot pump units because they are either below the elevation of the VGS line or they are below the hydraulic gradient of the VGS line. Because of the discovery that the VGS system could not be used effectively in much if not most of the service area, further evaluation of the VGS alternative for the whole service area was considered to be unproductive and was discontinued. Although the VGS system does not appear to offer a large-scale solution to sewering the service area, it could be implemented on a small scale to minimize costs. Since implementing the VGS system on a small scale will not significantly impact the sewer system total cost and because VGS could be used with any of the other alternative system, it is not included in the alternative evaluations which follow.

Variable-Grade Gravity Sewer System Unit Cost

Although the VGS system will not be further evaluated as an alternative, a discussion of the costs associated with the system is included herein to provide perspective of the costs of the VGS system in relation to the other sewer systems. While VGS pipe sizes are small and buried at shallow depths, pipeline construction costs also include the following:

- pavement saw cutting
- trench excavation
- pipe bedding and backfill material, placement, and compaction
- water and gas service connection repair
- pavement repair and replacement
- clean-outs
- testing and cleanup

Furthermore, as noted above, the size and depth of sewers will increase as population increases and topographical conditions become more adverse. Assuming an overage VGS unit price of $15.00/LF, rough capital cost estimate for a hypothetical VGS system in the Los Osos area would be as follows:

• 250,000 LF of sewer at $15.00/LF =	$ 3,750,000
• 1,500 on-lot pump units at $2,000/unit =	3,000,000
• 3,500 new VGS tanks at $1,000/tank =	3,500,000
• Five pump stations (see Table 6.4) =	2,000,000
• Force main interceptors (see Table 6.4) =	540,000
• Gravity main interceptor (see Table 6.4) =	520,000
• 20% contingency, 15% contractor's overhead and profit, 20% technical services =	8,700,000
• Total =	$22,000,000

The $22 million capital cost is comparable to the costs of the other alternatives.

COMBINATION SEWER SYSTEM

Combination System Description

The combination system is proposed to provide the least costly sewage collection system for the Los Osos service area through the use of two or more of the previously discussed alternative systems. The system which would be the least costly for any given area within the service area will be considered for that area. Areas exhibiting good drainage characteristics and high density, were considered as candidates for conventional gravity sewers. Variable-grade gravity sewers were considered for areas with good drainage characteristics but low density and/or high ground water. Areas with difficult topography, small drainage basins, low density and/or high groundwater were identified as candidates for pressure sewers.

Combination System Technical Feasibility

The previous discussions of technical feasibility for each alternative sewage collection system must be supplemented by a discussion of the feasibility of a combination of collection system types and the compatibility of those types.

In general, conventional gravity sewers and variable-grade gravity sewers should be compatible if the VGS system discharges to a conventional gravity sewer, but not vice versa. Raw sewage in a VGS system would clog the small-diameter, variable-grade sewers. It also may not be good practice to discharge pressure sewers to a VGS sewer because of the potential to surcharge the VGS system which may cause backflow to users connected to the VGS. Therefore, the VGS system is recommended for use in isolated sub-areas which will not drain other sub-areas with non-VGS systems and which are topographically suitable for VGS.

Conventional gravity sewers and pressure sewers should be compatible. Conventional gravity sewers may receive discharge from either STEP or grinder pump pressure sewers. Sewage pump stations in a conventional gravity sewer system should be able to pump into a pressure sewer interceptor if discharge pipe size is properly selected.

Another compatibility consideration of a hybrid sewage collection system is that STEP and VGS systems transport septic tank effluent while conventional gravity sewers and grinder pump systems transport raw sewage. A combination system with both septic tank effluent and raw sewage would affect treatment facilities design. Measures to mitigate the effect of septicity of septic tank effluent would be required in gravity sewers and sewage pump stations receiving STEP or VGS discharges and would also be required at the treatment facilities.

Pressure sewers have been used successfully in combination with gravity sewers in many of the existing pressure sewer installations. Pressure sewer systems are also capable of utilizing a combination of STEP units and grinder pump units although the two types are usually segregated into separate systems. One of the VGS projects in Ohio is proposed to be used in combination with a conventional gravity sewer system which will service industrial users (Reference 6.16). Therefore, within the previously discussed constraints, a combination sewer system should prove to be feasible.

Combination System Advantages

The primary advantage and goal of a combination system would be minimizing costs of the overall collection system. The use of each system in its most appropriate application should serve to enhance the inherent advantages of that system and minimize the disadvantages.

Combination System Disadvantages

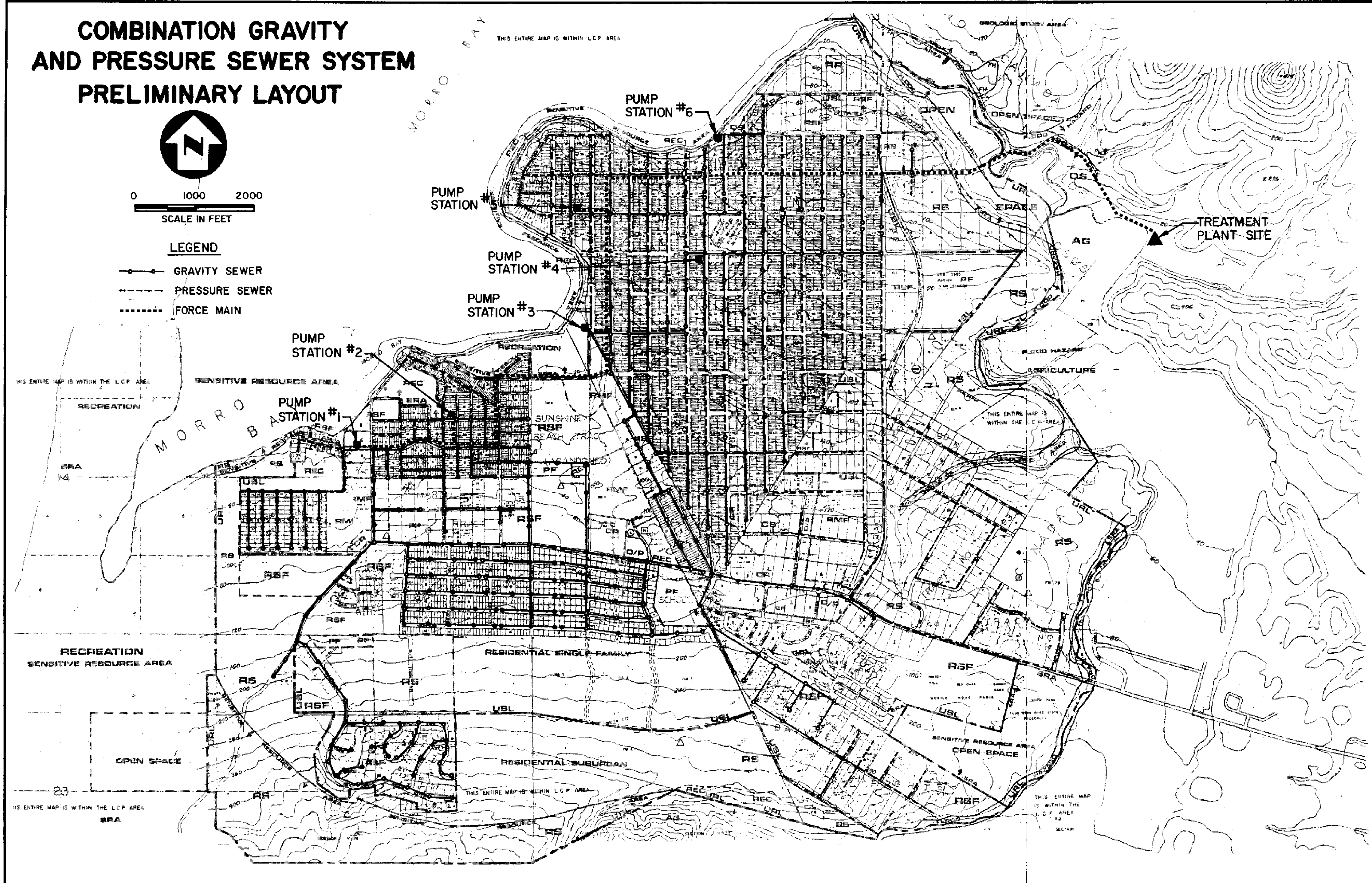
Due to diversity of O&M requirements for the various systems, O&M procedures could be complex and might result in higher costs. Different systems requiring different maintenance procedures and techniques at various intervals would necessitate coordination. The disadvantages of one system could carry over into other systems. An example of disadvantageous carry-over would be septic tank effluent septicity from a STEP or VGS system affecting the design of a raw sewage pump station.

Another disadvantage of a combination system would be that some users would require on-lot facilities such as septic tanks and pumps while other users would require only a sewer lateral.

Combination System Conceptual Design

The combination system was developed as a combination of STEP pressure sewers and conventional gravity sewers. The preliminary layout of the combination system is shown in Figure 6.6. Pressure sewers were proposed for those areas which had been previously identified as having high cost gravity sewers. The goal of identifying areas for pressure sewers was to reduce to the greatest degree possible the amount of high cost gravity sewers.

Pressure sewers were proposed for the low-lying, relatively flat area between the 20-foot contour and the shoreline because of high groundwater and deep gravity sewers. The low-lying area of gravity sewer Basin IV was also designated for pressure sewers because of high groundwater and deep gravity sewers. The southern portion of Baywood Park also employs pressure sewers in the combination sewer system to overcome deep gravity sewers resulting from the undulating terrain of this area. The Creekside area has pressure sewers because this is an isolated drainage basin with high groundwater and low density development. Based on the cost estimates for the alternative pressure sewer system given in Tables 6,9, 6.10, and 6.11, a system utilizing one STEP unit to serve two users with the unit located on one of the user's property was selected.


COMBINATION GRAVITY
AND PRESSURE SEWER SYSTEM
PRELIMINARY LAYOUT
0
1000
2000
SCALE IN FEET
LEGEND
GRAVITY SEWER
PRESSURE SEWER
FORCE MAIN
PUMP STATION #1
PUMP STATION #2
PUMP STATION #3
PUMP STATION #4
PUMP STATION #5
PUMP STATION #6
TREATMENT PLANT SITE
MORRO BAY
RECREATION
SENSITIVE RESOURCE AREA
RESIDENTIAL SINGLE FAMILY
RESIDENTIAL SUBURBAN
OPEN SPACE
AGRICULTURE
SUNSHINE BEACH TRACT
THIS ENTIRE MAP IS WITHIN THE L.C.P. AREA

In general, the remainder of the combination system consists of gravity sewers. Although not specifically considered, VGS sewers could be used in the combination system in those areas where they would be cost effective. Six pump stations are provided to transmit the flow to a point where it can flow by gravity to the treatment facilities. The alignment of the gravity main interceptor, the force main interceptor, and the location of pump stations generally correspond to that of the conventional gravity system. One of the conventional gravity system pump stations is eliminated in the combination system but two additional stations are required. The additional pump stations are necessary because the pressure sewers isolate gravity sewer areas. Pressure sewers discharge either to gravity sewers or to a pump station.

Design criteria for gravity sewers are given in Table 6.2. Design criteria for pressure sewers are given in Table 6.7. Design criteria for the combination system pump stations are given in Table 6.13. A summary of the combination sewer system components and a cost estimate for the system are given in Table 6.14.

TABLE 6.13

COMBINATION SEWER SYSTEM - PUMP STATION PRELIMINARY DESIGN CRITERIA

Pump Station	Design PWWF		TDH	Total[a] HP Reqd.	Pump Type	Standby Power	Speed Control	Odor Control
	mgd	gpm	(ft)					
1	1.0	700	160	40	dry pit	permanent	variable	yes
2	1.0	700	140	35	dry pit	permanent	variable	yes
3	1.3	900	120	40	dry pit	permanent	variable	yes
4	1.1	760	90	25	dry pit	permanent	variable	yes
5	0.5	350	110	15	submersible	portable	constant	no
6	0.4	280	110	10	submersible	portable	constant	no

[a]Assuming 70 percent overall pump efficiency.

TABLE 6.14

COMBINATION SEWER SYSTEM - PRELIMINARY CAPITAL COST ESTIMATE

Item	Quantity	Unit Cost ($)	Total Cost ($)
Gravity Sewers			
6-inch diameter	129,000 lf	25.00/lf	3,225,000
8-inch diameter	18,000 lf	26.00/lf	468,000
10-inch diameter	7,000 lf	26.00/lf	182,000
Pressure Sewers			
3-inch diameter	87,000 lf	16.00/lf	1,392,000
4-inch diameter	2,000 lf	17.00/lf	34,000
6-inch diameter	2,000 lf	21.00/lf	42,000
8-inch diameter	7,000 lf	23.00/lf	161,000
Force Main Interceptors			
6-inch diameter	1,100 lf	21.00/lf	23,000
8-inch diameter	2,900 lf	23.00/lf	67,000
12-inch diameter	3,800 lf	33.00/lf	125,000
16-inch diameter	5,800 lf	39.00/lf	226,000
Gravity Interceptor			
18-inch diameter	8,700 lf	55.00/lf	479,000
Pump Station No. 1	--	--	400,000
Pump Station No. 2	--	--	400,000
Pump Station No. 3	--	--	400,000
Pump Station No. 4	--	--	300,000
Pump Station No. 5	--	--	150,000
Pump Station No. 6	--	--	150,000
Cluster System Pump Stations	3	50,000 ea	150,000
Gravity Sewer Laterals			
4-inch diameter	174,000 lf	15.00/lf	2,610,000
On-Lot Pump Facilities	1,100	2,000 ea	2,200,000
SUBTOTAL			13,184,000
Contingency (20%)			2,637,000
SUBTOTAL			15,821,000
Contractor's Overhead & Profit (15%)			2,373,000
SUBTOTAL (Construction Costs)			18,194,000
Technical Services			
Basic Design Services (5.8%)			1,055,000
Right-of-Way Easement Acquisition Services (3%)			546,000
Other Technical Services (12%)[a]			2,183,000
TOTAL			21,978,000

[a]Includes engineering, legal, financial, and administrative services.
NOTE: ENR CCI 5180

Combination Sewer System Unit Costs

Unit costs for the combination sewer system were taken from the conventional and pressure sewer system unit costs. As with the conventional gravity sewer system, gravity sewer unit costs were based on the quantity of high cost and low cost sewers. Pump station cost estimates were derived on the same basis as the conventional and pressure sewer pump station cost estimates.

Combination System Operation and Maintenance

Operation and maintenance activities for the combination system include gravity sewer cleaning, pressure sewer maintenance, pump station operation and maintenance, and STEP on-lot facilities operation and maintenance. The same asssumptions regarding O&M for the conventional and pressure sewer systems apply to the combination system.

Table 6.15 shows the estimated annual O&M costs for the first year of operation. For the purposes of this study, it was assumed that two workers with a high pressure sewer cleaner would be responsible for gravity sewer and pressure sewer maintenance. A separate two-person crew would be responsible for pump station and on-lot facilities maintenance. A fifth worker with a pumper truck would be responsible for pumping septage from STEP interceptor tanks.

TABLE 6.15

COMBINATION SEWER SYSTEM
ESTIMATED FIRST YEAR O&M COSTS

Item	Estimated Cost
Labor - 5 employees at $30,000/year	$150,000
Materials	25,000
Energy	
Pump Stations 1 thru 6	19,000
STEP units	6,000
TOTAL	$200,000

PRESENT-WORTH ANALYSIS

A present-worth analysis was performed for the conventional gravity sewer, pressure sewer, and conventional sewer and pressure sewer combination alternatives. Results of the present-worth analyses are presented in Tables 6.16 through 6.20. Present worth for each alternative was developed based on a 20-year period and a discount rate of 8-3/8 percent. All project costs within the 20-year study period were considered.

Capital Costs

Capital costs for each alternative were developed previously in this chapter and are shown in Tables 6.5, 6.9, 6.10, 6.11, and 6.14. Capital costs include construction, engineering, legal, financial and administrative costs necessary to implement the project and were assumed to occur at year zero. Capital costs are based on an ENR Construction Cost Index of 5180, which is an average of the May 1985 CCI for Los Angeles and San Francisco. Deferred capital costs for the pressure sewer and combination sewer systems represent the cost of future STEP on-lot pump units required to accommodate future development of existing lots. Capital costs for future on-lot facilities were estimated based on a 2 percent annual growth rate over the 20-year period. The 20 percent allowance for technical services is not included in the capital cost for future on-lot facilities.

TABLE 6.16

CONVENTIONAL GRAVITY SEWER SYSTEM
PRESENT WORTH ANALYSIS SUMMARY

Item	Present Worth
Capital Cost (ENR CCI 5180)	$24,619,000
Operation and Maintenance	
Labor	1,146,000
Materials	339,000
Energy	315,000
SUBTOTAL	$26,419,000
Salvage Value	(2,000,000)
TOTAL Present Worth	$24,419,000

TABLE 6.17

PRESSURE SEWER SYSTEM - PRESENT WORTH ANALYSIS SUMMARY
ON-LOT FACILITIES ALTERNATIVE A - ONE UNIT PER USER

Item	Present Worth
Capital Cost (ENR CCI 5180)	$23,431,000
Deferred Capital Costs	3,292,000
Operation and Maintenance	
Labor	2,006,000
Materials[a]	653,000
Energy	263,000
Treatment Facilities Cost Credit	(300,000)
SUBTOTAL	$29,345,000
Salvage Value	(1,605,000)
TOTAL Present Worth	$27,740,000

[a]Includes replacement costs for pump units which fail prior to expiration of useful life.

TABLE 6.18

PRESSURE SEWER SYSTEM - PRESENT WORTH ANALYSIS SUMMARY
ON-LOT FACILITIES ALTERNATIVE B - ONE UNIT PER TWO USERS

Item	Present Worth
Capital Cost (ENR CCI 5180)	$16,365,000
Deferred Capital Costs	1,646,000
Operation and Maintenance	
Labor	1,719,000
Materials[a]	496,000
Energy	263,000
Pump Replacement (after useful life)	550,000
Treatment Facilities Cost Credit	(300,000)
SUBTOTAL	$20,739,000
Salvage Value	(1,171,000)
TOTAL Present Worth	$19,568,000

[a]Includes replacement costs for pump units which fail prior to expiration of useful life.

TABLE 6.19

PRESSURE SEWER SYSTEM - PRESENT WORTH ANALYSIS SUMMARY
ON-LOT FACILITIES ALTERNATIVE C - ONE UNIT IN R/W PER TWO USERS

Item	Present Worth
Capital Cost (ENR CCI 5180)	$23,670,000
Deferred Capital Costs	1,646,000
Operation and Maintenance	
Labor	1,719,000
Materials[a]	496,000
Energy	263,000
Pump Replacement (after useful life)	550,000
Treatment Facilities Cost Credit	(300,000)
SUBTOTAL	$28,044,000
Salvage Value	(1,622,000)
TOTAL Present Worth	$26,422,000

[a]Includes replacement costs for pump units which fail prior to expiration of useful life.

TABLE 6.20

COMBINATION SEWER SYSTEM
PRESENT WORTH ANALYSIS SUMMARY

Item	Present Worth
Capital Cost (ENR CCI 5180)	$21,978,000
Deferred Capital Costs	673,000
Operation and Maintenance	
Labor	1,433,000
Materials[a]	516,000
Energy	400,000
Pump Replacement (after useful life)	279,000
Treatment Facilities Cost Credit	(100,000)
SUBTOTAL	$25,179,000
Salvage Value	(1,678,000)
TOTAL Present Worth	$23,501,000

[a]Includes replacement costs for pump units which fail prior to expiration of useful life.

Operation and Maintenance Costs

Operation and maintenance costs were divided into labor, materials, energy and replacement costs. Labor requirements were developed previously in this chapter. Labor costs are estimated based on a total cost of $30,000 per year per worker. A common base cost for materials and supplies of $25,000 per year was assumed for all alternatives. Additional materials costs for the pressure sewer and combination sewer systems were included to account for repair of on-lot pumps at a failure rate of 2 percent a year except during the first 5 years when it was assumed that the pumps would be warranted. Materials cost also includes initial capital costs for maintenance vehicles.

Energy Costs

Energy costs for sewage pumping were estimated based on initial requirements escalated at 2 percent a year to correspond to the projected population growth rate over the 20-year period. (This rate is less than the year 2000 2.4 percent growth projection due to the longer time span.) The unit cost used to estimate energy costs was $0.08/kWh. The energy unit cost was also escalated at an annual rate of 4 percent to account for future increases in the cost of energy. Initial energy costs were provided in Tables 6.6, 6.12, and 6.15. Energy costs include energy for main line pump stations and STEP on-lot units. Grinder pump on-lot energy costs would be higher than STEP units.

Replacement Costs

Replacement costs for STEP on-lot pumps after expiration of their useful are included in the present worth of the pressure sewer and combination sewer alternatives. A useful life of 20 years was assumed for a typical STEP pump serving one user. However, for pumps which must serve two users, a life span of 10 years was assumed, thereby resulting in replacement requirements within the 20-year study period. Replacement costs were estimated based on a STEP pump cost of $500 per unit. (Replacement unit cost for a grinder pump would be higher than for a STEP unit.) Replacement costs for pumps serving two users therefore occur in year 10 and year 20. Replacement costs for 10-year lifespan pumps also occur in years 11 through 19 to account for future on-lot units installed in years 1 through 10 assuming an annual growth rate of 2 percent.

Treatment Facilities Cost Credit

As previously discussed in this chapter, the use of pressure sewers will reduce flow and load requirements and, therefore, costs at the treatment facilities. Reduced treatment costs are a result of reduced flow due to the lack of I/I in pressure sewers and reduced BOD and suspended solids loads of septic tank effluent. While some of the reduced costs will be offset by costs of odor and corrosion control associated with effluent septicity and by requirements for septage receiving facilities, alternative sewer systems which utilize pressure sewers are due a cost credit associated with cost reduction at the treatment facilities.

The treatment facilities cost credit is the difference between the cost of the facilities without the use of pressure sewers and the cost of the facilities with the use of pressure sewers. The cost of the treatment facilities with a pressure sewer collection system was estimated by applying a cost-reduction factor determined as the ratio of the design flow with pressure sewers and the design flow without pressure sewers to the two-thirds power. The cost-reduction factor was applied only to those process units affected by a reduction in flow or load. This method of calculating the savings associated with reduced I/I has been proven accurate in I/I reduction cost-effectiveness studies which ES has performed, particularly studies for the Monterey Regional Water Pollution Control Agency (Reference 6.16). Additional treatment costs associated with the use of pressure sewers, including odor control, corrosion control, and septage receiving, were then added to the cost of the treatment facilities for a pressure sewer system. The difference between the treatment facilities cost with conventional gravity sewers and with pressure sewers is shown as a cost credit in the present worth summary.

Salvage Value

Salvage value of all facilities was considered based on a linear depreciation of the facility construction cost over its useful lifespan.

The salvage value given in the present worth analysis summary tables is the present worth of the depreciated value of the facilities at the end of the 20-year study period. Salvage value was treated as a present worth credit in order to take into account variations in useful lives of different components in the alternative systems. The salvage value in this analysis represents the unused portion of the value of the system, or "implied" salvage value, at the end of the study period.

The assumed useful life of all pipelines, interceptor tanks, and pump station structures was assumed to be 40 years. The useful life of mechanical equipment in pump stations and STEP pump units was assumed to be 20 years. It was assumed that the initial construction cost of mechanical equipment in the pump station and STEP units was 25 percent of the total pump station construction cost. Costs for mechanical equipment having a shorter useful life span than 20 years are considered as replacement costs if replacement is required during the 20-year period as dicussed above.

Discussion

Table 6.21 summarizes the total present worth of the alternative systems. Table 6.21 reveals that the pressure sewer system Alternative B, which proposes one on-lot unit per two users, has the lowest present worth. In spite of O&M costs nearly twice as high as the conventional gravity sewer alternative, substantial capital cost savings allow this pressure sewer alternative present worth to be about 20 percent less than the conventional alternative.

However, it should be made clear that the pressure sewer savings was realized only through the assumption that two homes would share a single on-lot pump unit, the feasibility of which is discussed in the following section. If an on-lot pump unit is provided for each user (Alternative A), the capital costs alone, including initial and deferred, would be over $26 million which is approximately $2 million greater than the conventional system total present worth. O&M costs would also be expected to rise if one STEP unit is assumed for each user. Costs associated with placing the pressure system pump units in the County right-of-way (Alternative C) eliminate the capital cost saving advantage of the pressure sewer system.

TABLE 6.21

ALTERNATIVE TOTAL PRESENT WORTH SUMMARY

Alternative System	Total Present Worth ($1,000)	Percent of Conventional (%)
Conventional Gravity Sewers	24,419	100
Pressure Sewers		
Alternative A - one unit per user	27,740	114
Alternative B - one unit per two users	19,568	80
Alternative C - one unit in R/W per two users	26,422	108
Combination System	23,501	96

While the combination pressure sewer/gravity sewer system has a lower present worth than the conventional gravity sewer system, the combination system did not achieve the major cost reduction expected by eliminating troublesome gravity sewer sections. Instead, the present worth of the combination system is between the present worth of the pressure sewer system and gravity sewer system in rough proportion to the percentage of each.

Energy costs for the pressure sewer alternatives are less than for the conventional system. This is due to the fact that elevation head is preserved in the pressure system while sewage in the conventional system flows to the lowest point in the drainage basin before pumping occurs. Also in the conventional system, sewage from Basin III is pumped into Basin II where it is again pumped into the main interceptor.

QUALITATIVE ANALYSIS

The advantages and disadvantages of each alternative sewer system are discussed in detail throughout this chapter. In general, the conventional system costs more to build but is easier to maintain, while the alternative systems cost less to build and more to maintain and have a higher potential for future operational problems. If a substantial cost savings is to be realized through the use of alternative sewer systems, users must be willing to live with increased inconveniences.

Since it has been determined that a pressure sewer system will not be cost effective unless one on-lot pump unit is shared between two users, the public acceptability of the shared pump unit concept must be evaluated prior to serious consideration of the pressure sewer alternative. Several issues arise when the sharing of on-lot facilities is considered. Given a choice, most homeowners may prefer not to have a pump unit on their lot, especially when their neighbor does not have one. A method would have to be devised to compensate the homeowner with the on-lot unit for actual or perceived expenses associated with electrical power consumption of the unit (power supply will be from the house electrical connection), granting of easement for the unit, possible decreased property values, easement for the neighbor's connection, failure light monitoring responsibility and so forth. Compensation could be in the form of a two-tier assessment, a lower assessment for those lots with a pump unit and a higher assessment for those lots without a pump unit. The administrative problem of determining who gets an on-lot unit must be resolved.

Another consideration of the use of alternative collection systems is that no alternative system has been proposed for a community of the size and density of population of CSA No. 9. Of course, the fact that alternative systems have not been used for communities the size of CSA No. 9 does not mean that they can not be used; however, it does seem to indicate that alternative systems generally have been considered feasible for only small rural communities.

CONCLUSIONS AND RECOMMENDATIONS

This study has determined that a pressure sewer system with one on-lot unit per two users should be less costly than a conventional gravity sewer system. However, there are significant uncertainties associated with the implementation of a pressure sewer system which are difficult to quantify in terms of cost. These uncertainties include the potential for maintenance costs to be higher than anticipated (as with the Ventura County systems), the ability to successfully implement a pressure system of the scale proposed, administration of on-lot pump unit location and maintenance, and public acceptability.

With these uncertainties come risks not associated with a conventional system. In those communities where pressure sewers have been selected over conventional sewers for new sewer systems, the costs savings have clearly justified the additional risk. The cost of a conventional system was typically at least several times as costly as the alternative pressure sewer system in most of these cases. Increased federal funding from Innovative and Alternative System grant monies was provided for all of these projects and thus added an additional financial incentive to counter the uncertainties of a pressure sewer system. The possibility of obtaining Innovative and Alternative funding for this project is presently unknown.

It is therefore concluded that the cost savings afforded by a pressure sewer system with one on-lot pump unit per two users do not justify the uncertainties associated with such a system for this particular application. A conventional gravity sewer system with limited use of pressure sewers in the most troublesome areas is the recommended sewage collection system for this project. Alternative sewer systems cannot provide cost benefits of a conclusive magnitude over a conventional system, primarily because of the high population density and relatively large population of the service area and because most of the terrain of the service area is conducive to a gravity sewer system.

The use of one gravity sewer lateral to serve two users where topographically appropriate could reduce the cost of the conventional system to a level comparable to the pressure sewer system. The actual cost implications of this arrangement can be evaluated during final design when it will be possible to quantify cost savings based on the actual number of locations where it would be possible to use one lateral for two users.

CHAPTER 7

DEVELOPMENT AND EVALUATION OF ALTERNATIVE TREATMENT SYSTEMS

CHAPTER 7

EVALUATION OF ALTERNATIVE TREATMENT SYSTEMS

SITE LOCATION

The Los Osos Wastewater Treatment Facility will be located at the Turri Road site recommended in the "Los Osos-Baywood Park Phase II Facilities Planning Study" (Reference 7.1). As shown on Figure 6.1, the site lies approximately one mile from the nearest residential area, on the east side of Baywood Park. The site is currently undeveloped except for partial use as pasture. Prevailing winds are primarily from the west and northwest, and thus, if occasional treatment plant odors exist, they will be directed away from residential areas. At elevation 20 m.s.l., the site is suitable to receive gravity flow from the collection system, thus eliminating the need for a raw sewage pump station to lift flow to the treatment plant. The site borders the 100-year flood plain of Los Osos Creek, as noted on the 1965 USGS "Map of Flood Prone Areas" (Reference 7.2). Some flood protection may be required.

TREATMENT ALTERNATIVE EVALUATION CRITERIA

To determine the treatment process most suitable for Los Osos, each alternative was evaluated with respect to environmental impact, feasibility and performance factors, labor, chemicals and energy cost, construction cost, and total present worth. All processes, of course, must be designed to achieve consistently the discharge requirements set forth in Chapter 4.

Cost Analysis

The cost analysis provides for a consistent and systematic comparison of alternatives in order to identify the alternative which will result in the lowest total costs over a twenty-year period, while meeting the specified goals and objectives. The most cost-effective alternative is the waste treatment/management system determined from the analysis to have the lowest 20-year present worth cost without overriding environmental and/or social impacts.

Capital and operations and maintenance (O&M) costs were obtained from the EPA publications "Construction Costs for Municipal Wastewater Treatment Plants: 1973-1982" and "Innovative and Alternative Technology Assessment Manual" (February 1980) and adjusted based on previous Engineering-Science projects. These costs are updated to May 1985 dollars. The ENR Construction Cost Index of 5180 used is an average of the San Francisco and Los Angeles May 1985 indices. Costs for secondary treatment and nitrogen removal for Alternatives 1 and 2 are based on system quotations and preliminary quantities of concrete, earthwork and other materials, priced according to Engineering-Science experience.

cost basis

The cost analysis for the screening of alternatives is based on the following criteria:

- Annual rate on capital of 8-3/8 percent.
- Cost evaluation period of 20-years.
- Capital costs are developed for comparative purposes based on construction costs for May 1985 (ENR CCI 5180).
- Selection of the cost-effective alternative based on capital construction costs plus present worth of operation and maintenance (O&M) costs over a twenty-year period.

The cost estimates presented are order of magnitude level estimates suitable for comparison of alternatives and are not adequate for detailed financial planning in accordance with the authorized level of effort for this Phase I study.

DESCRIPTION OF ALTERNATIVES

General

Three alternatives for nitrogen removal will be evaluated herein:

- Continuous Loop Reactor
- Sequencing Batch Reactors
- Physical-Chemical Treatment

Each process will be described subsequently. The first two alternatives are biological and have the following common unit operations:

- Headworks (screening, grit removal, Parshall flume, grit removal, and septage receiving)
- Sludge drying beds
- Tertiary treatment facilities
- Operations facilities

The physical-chemical process is completely different and non-biological and is therefore evaluated separately.

Alternative 1. Continuous Loop Reactor

Plant influent flows through a mechanically cleaned bar screen and Parshall flume to a grit removal tank. Septage is received separately and is screened before entering the same pretreatment facilities. The screenings and grit removed are trucked to a local landfill.

The screened and degritted wastewater enters a continuous loop reactor (CLR) (also known as an oxidation ditch) consisting of three or four connected concentric loops. Rotating discs or similar aeration devices provide oxygen needed for biological oxidation and nitrification as well as energy needed to circulate the flow. The CLR, operating in the extended aeration mode of the activated sludge process, removes nitrogen and biochemical oxygen demand (BOD_5) while simultaneously stabilizing solids. The CLR exposes the wastewater to a series of aerobic and anoxic cycles (aerated and non-aerated) to convert ammonia to nitrogen gas. Following the CLR, two identical secondary clarifiers separate liquid and solids.

Tertiary treatment consists of flocculation and filtration to remove secondary effluent solids. The chemicals that will likely be used are alum (aluminum sulfate) and polymer (polyelectrolyte coagulant aid). Disinfection at a chlorine contact tank to reduce bacteria and viruses to the required discharge standard is followed by dechlorination with sulfur dioxide and effluent pumping to off-site disposal.

Waste activated sludge is dewatered at sludge drying beds. Sludge digestion is not necessary due to the stabilization achieved in the CLR. During wet weather, sludge will be diverted to a sludge lagoon and held until appropriate weather for drying is available. The dried sludge soilds could be made available to local residents as a soil amendment; however, some analyses of the actual sludge produced would probably be required by regulatory agencies before allowing the general public to reuse the sludge. For planning purposes, it is assumed that all sludge will be trucked to a landfill.

The process flow diagram for Alternative 1 is shown in Figure 7.1.

Proprietary Process Considerations

A review of the following continuous loop reactor processes was conducted to select one appropriate for Los Osos (Reference 7.3):

- Carrousels using vertical surface aerators,
- Jet aeration channels using jet aerators,
- Orbal systems using rotor-mounted perforated disks,
- Single loop plants using brush or cage aerators,
- Barrier ditches using submerged or surface turbine and draft tubes, and
- Bardenpho system with anoxic tanks before and after the CLR.

The orbal process, a proprietary CLR process of Envirex, Inc., was selected for consideration in this preliminary study. Other CLR processes could also be appropriate for Los Osos, and CLR process selection does not affect the comparison of Alternatives 1, 2, and 3.

Alternative 2. Sequencing Batch Reactors

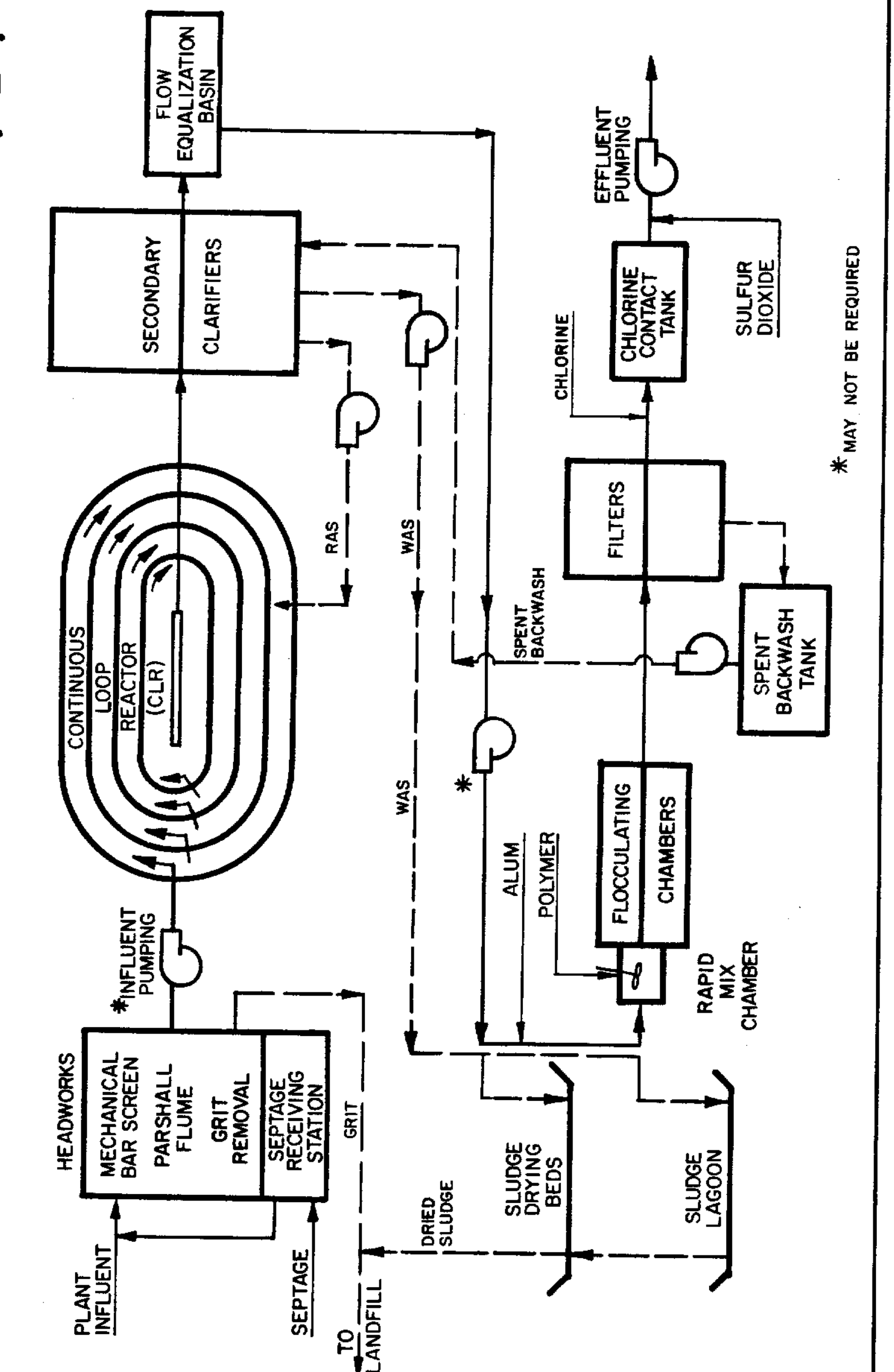
Plant influent flows through a mechanically cleaned bar screen and Parshall flume to a grit removal tank. Septage is received separately and is screened before entering the same pretreatment facilities. The screenings and grit removed are trucked to a local landfill.

The sequencing batch reactor (SBR) process consists of parallel batch activated sludge systems operating in the extended aeration mode. SBRs achieve biological oxidation, nitrogen removal, solids stabilization, and solids separation all in the same tank, although multiple

FIGURE 7.1

PROCESS FLOW DIAGRAM

TREATMENT ALTERNATIVE 1. CONTINUOUS LOOP REACTOR (CLR)



ENGINEERING-SCIENCE, INC.

tanks are used, depending on the flow. These processes occur in each SBR tank during successive fill, react, anoxic mix, settle, and draw operating modes as shown in Figure 7.2. SBRs have considerable flexibility in that mixing time, aeration time, and time in each operating mode are easily adjusted to optimize performance. Operating modes, such as fill and react or anoxic mix and settle, may be combined to shorten total batch time. Aeration is achieved by jet aeration or coarse bubble diffusion equipment.

Although SBR technology is relatively new in the United States, over 35 plants are operating successfully in Australia and Japan, some since 1976. The first SBR facility in the U.S. started operation in 1980 and at least 20 others are now either operating or under construction. The batch activated sludge process used in SBRs has been used, under various names, for about 70 years (Reference 7.4).

In Alternative 2, the SBR system replaces the CLR and secondary clarifiers used in Alternative 1; all other elements of the flow stream remain unchanged. The process flow diagram for Alternative 2 is shown in Figure 7.3.

Proprietary Process Considerations

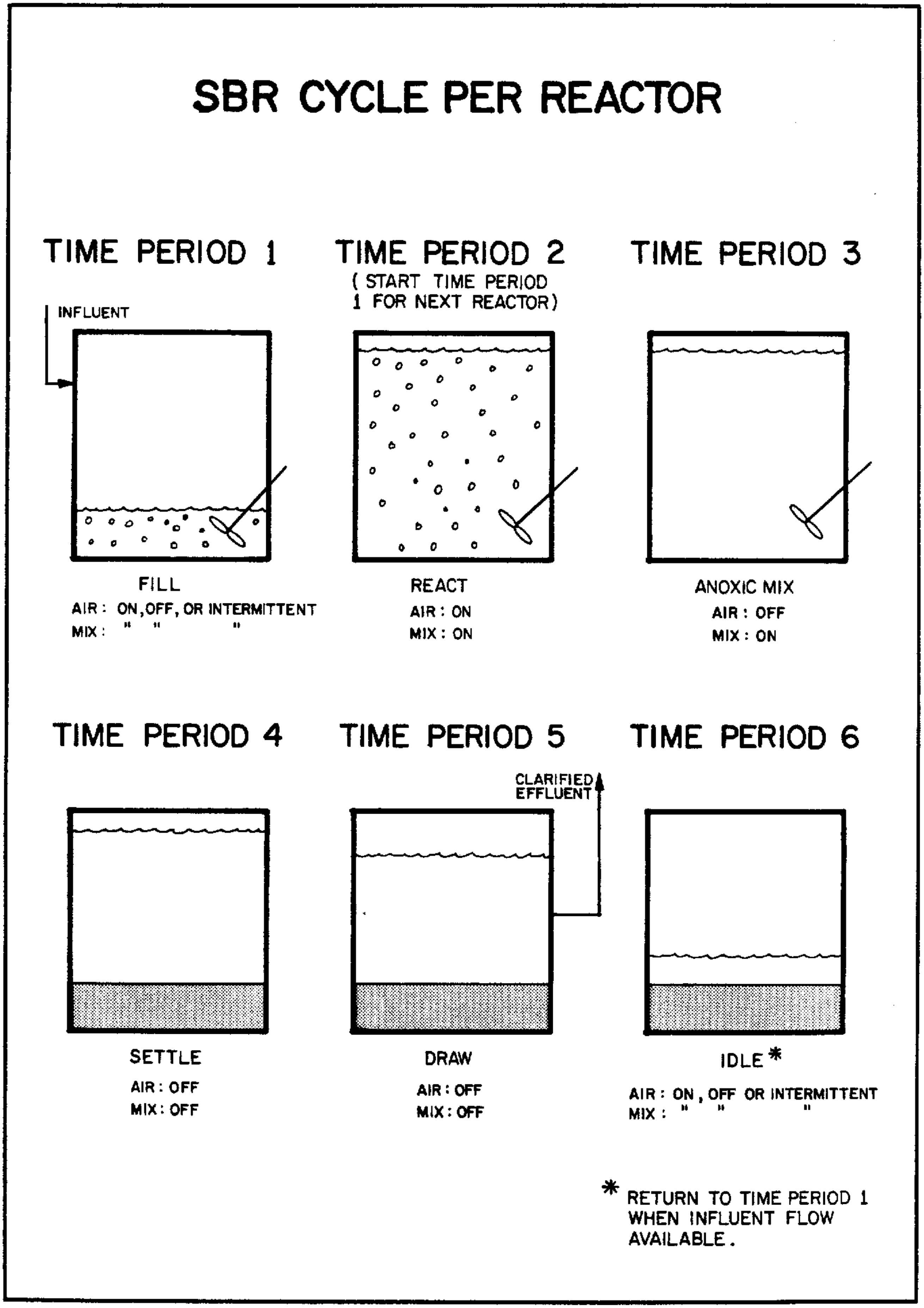
Several manufacturers hold U.S. patents for variations of the SBR process. These include the SBR system of the Fluidyne Corporation and the Intermittent Cycle Extended Aeration System (ICEAS) of Austgen Bio-jet Wastewater Systems, Inc.

Alternative 3. Physical-Chemical Treatment

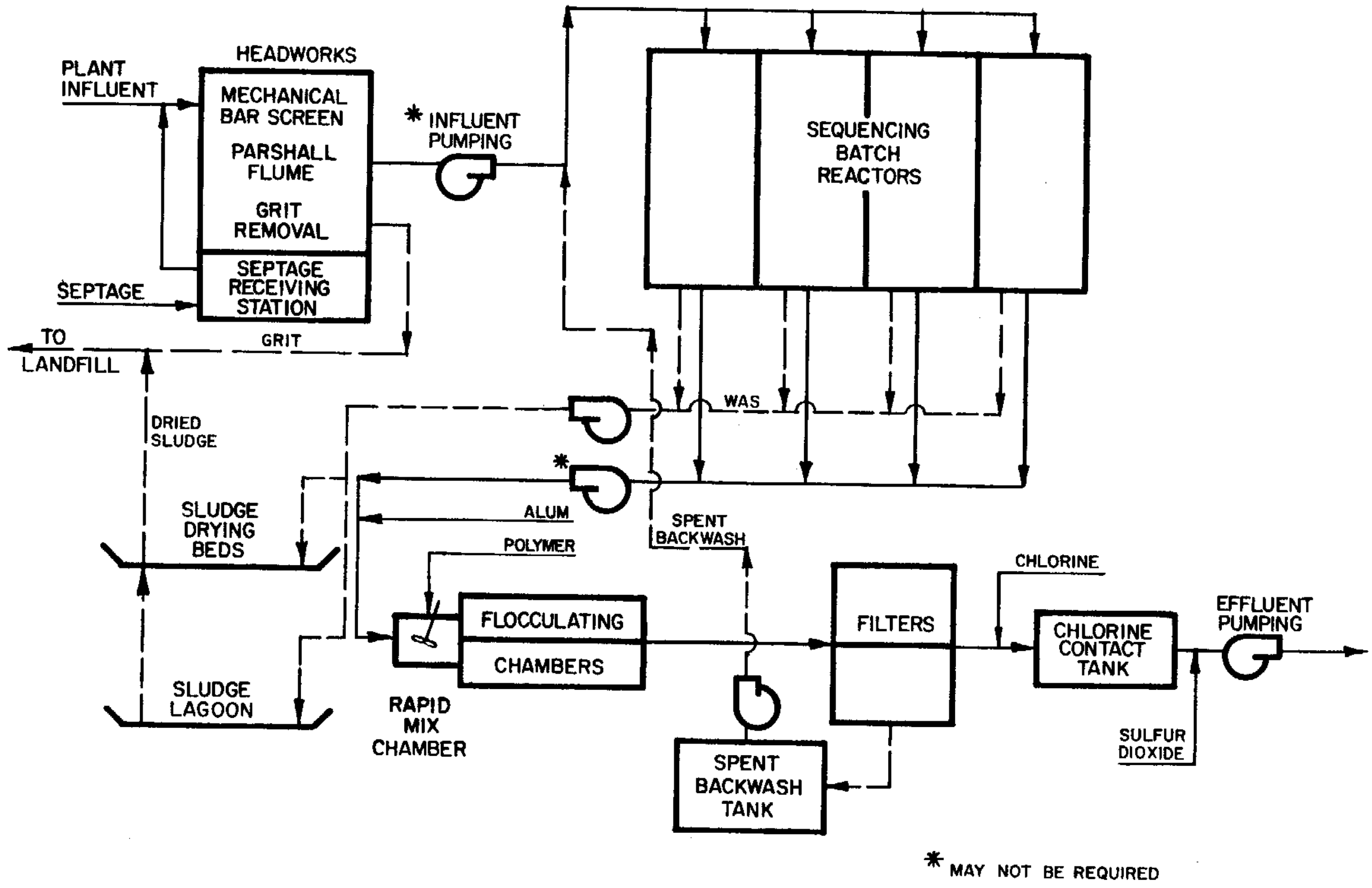
Physical-Chemical Treatment Processes

Plant influent flows through a mechanically cleaned bar screen and Parshall flume to a grit removal tank. Septage is received separately and is screened before entering the same pretreatment facilities. The screenings and grit removed are trucked to a local landfill.

The screened and degritted wastewater then flows to one of several alternative chemical and physical pollutant-removing unit operations. The first step in all these processes is high dose chemical coagulation


SBR CYCLE PER REACTOR
TIME PERIOD 1
INFLUENT
FILL
AIR : ON, OFF, OR INTERMITTENT
MIX : " " "
TIME PERIOD 2
(START TIME PERIOD 1 FOR NEXT REACTOR)
REACT
AIR : ON
MIX : ON
TIME PERIOD 3
ANOXIC MIX
AIR : OFF
MIX : ON
TIME PERIOD 4
SETTLE
AIR : OFF
MIX : OFF
TIME PERIOD 5
CLARIFIED EFFLUENT
DRAW
AIR : OFF
MIX : OFF
TIME PERIOD 6
IDLE *
AIR : ON, OFF OR INTERMITTENT
MIX : " " "
* RETURN TO TIME PERIOD 1 WHEN INFLUENT FLOW AVAILABLE.

PROCESS FLOW DIAGRAM
TREATMENT ALTERNATIVE 2. SEQUENCING BATCH REACTORS (SBRs)



and precipitation (clarification). The chemical coagulation and precipitation process removes BOD and suspended solids, but is not sufficient to achieve the required discharge levels of 10-10 mg/l BOD_5-suspended solids, or ammonia removal. The flow must be subsequently treated by filtration and activated carbon adsorption (considered herein) or, alternatively, preceded by filtration and/or a biological process (not considered herein).

Non-biological methods of removing nitrogen from wastewater include the following:

1. Breakpoint chlorination,

2. Ammonia removal by selective ion exchange, and

3. Ammonia stripping.

The first method, breakpoint chlorination, uses superchlorination to oxidize ammonia nitrogen to nitrogen gas. At first, the chlorine oxidizes any readily oxidizable organic matter, after which the chlorine-ammonia reaction takes place, ultimately producing nitrogen gas and hydrochloric acid. The acidity produced by the reaction must be neutralized by lime addition, resulting in the need for subsequent solids handling. Dechlorination is required to reduce the toxicity of the effluent. Breakpoint chlorination involves substantial operating expenses for chemical additions and lime sludge handling. It is effective when a near-zero effluent nitrogen level is required.

The second method, ammonia removal by ion exchange, utilizes the high selectivity of certain ion exchange materials for the ammonia ion. Clinoptilolite, a naturally occurring zeolite, has been used for this purpose. Drawbacks of the ion exchange system include its high capital cost as compared with the other two systems and problems in handling regeneration backflow (five percent of the flow) which may contain 300 mg/l ammonia.

The third method, ammonia stripping, uses a desorption process in which ammonia gas dissolved in the wastewater is stripped by air. The process basically consists of bringing small drops of wastewater in contact with large amounts of air so that, by reducing the partial pressure, ammonia gas is forced to leave the water phase and enter the air. To be stripped, ammonia must be in its un-ionized molecular form, NH_3, rather than in its ionic form, NH_4^+. This is accomplished by adding lime to raise the pH of the wastewater to between 10 and 11. Lime also coagulates and precipitates other sewage components, including phosphate. After the ammonia stripping phase, the pH is readjusted to neutral by addition of carbon dioxide gas. Drawbacks of ammonia stripping include cost of chemicals, scaling problems, and lime sludge handling.

<u>Selection of the Physical-Chemical Process for Alternative 3</u>

Effluent requirements and cost data indicate that the most suitable physical-chemical process for Los Osos consists of pretreatment, chemical addition, coagulation and clarification, ammonia stripping, filtration, and activated carbon adsorption. The process flow diagram for Alternative 3 is shown in Figure 7.4.

EVALUATION OF ALTERNATIVES

Preliminary capital cost estimates were developed for each of the alternatives and are presented in Tables 7.1, 7.2 and 7.3. A summary of the economic analysis is presented in Table 7.4. The evaluation of alternatives is summarized as follows:

- There are no apparent overriding adverse environmental impacts present for any of the alternatives.
- All three of the selected alternatives can meet the effluent quality requirements.
- SBRs have the lowest land requirements of the alternatives considered.
- SBRs have the highest operational flexibility of the three alternatives. The SBR process allows greater control over the nitrification and denitrification process than the CLR.
- The CLR process has a good record for reliability of secondary treatment, based on numerous U.S. facilities. The SBRs also have a good record for reliability of secondary treatment, but this is based on a limited data base available from the few SBR

FIGURE 7.4

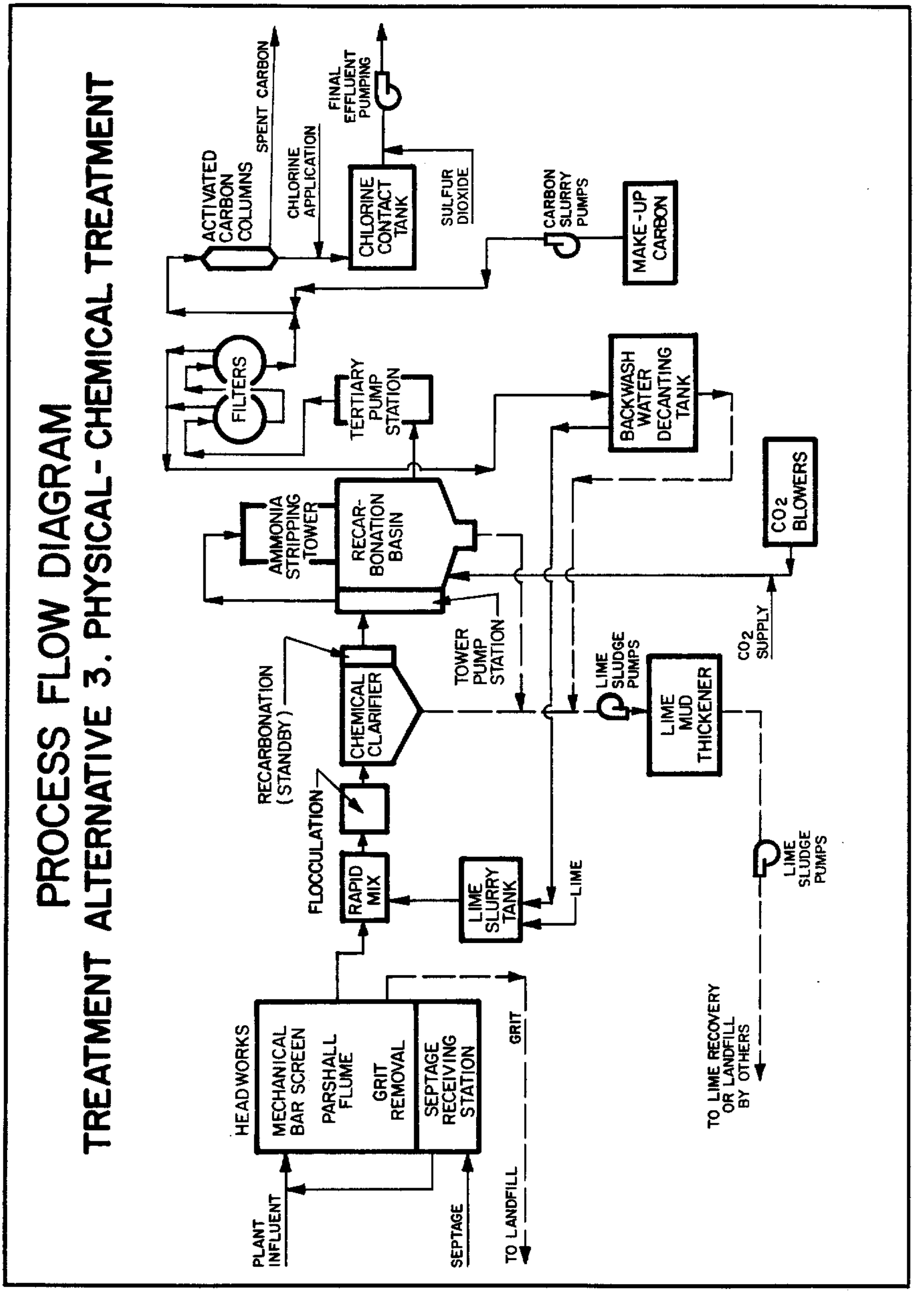

PROCESS FLOW DIAGRAM
TREATMENT ALTERNATIVE 3. PHYSICAL - CHEMICAL TREATMENT
PLANT INFLUENT
SEPTAGE
TO LANDFILL
HEADWORKS
MECHANICAL BAR SCREEN
PARSHALL FLUME
GRIT REMOVAL
SEPTAGE RECEIVING STATION
GRIT
RAPID MIX
FLOCCULATION
LIME SLURRY TANK
LIME
RECARBONATION (STANDBY)
CHEMICAL CLARIFIER
TOWER PUMP STATION
LIME SLUDGE PUMPS
LIME MUD THICKENER
LIME SLUDGE PUMPS
TO LIME RECOVERY OR LANDFILL BY OTHERS
AMMONIA STRIPPING TOWER
RECAR-BONATION BASIN
CO_2 SUPPLY
CO_2 BLOWERS
TERTIARY PUMP STATION
FILTERS
BACKWASH WATER DECANTING TANK
ACTIVATED CARBON COLUMNS
SPENT CARBON
CHLORINE APPLICATION
CHLORINE CONTACT TANK
SULFUR DIOXIDE
FINAL EFFLUENT PUMPING
CARBON SLURRY PUMPS
MAKE-UP CARBON

TABLE 7.1

PRELIMINARY CONSTRUCTION COST ESTIMATE
ALTERNATIVE 1. CONTINUOUS LOOP REACTOR

Item Description	Estimated Cost (1985 $)[a]
1. Headworks. Mechanical bar screen, manual bar screen, Parshall flume, grit removal tank, and septage receiving station.	160,000
2. Secondary Treatment and Nitrogen Removal. CLR, 2 secondary clarifiers, flow equalization, and associated pumps, valves, piping, and controls.	1,480,000
3. Tertiary Treatment. Rapid mix and flocculation chambers, filters and spent backwash facilities, chlorination and chlorine contact tank, dechlorination.	1,660,000
4. Ancillary Facilities. Yard piping, electrical and instrumentation, control building, and laboratory.	1,130,000
5. Sludge Disposal. Sludge lagoon and drying beds.	300,000
SUBTOTAL	4,730,000
Contingency (20%)	950,000
SUBTOTAL	5,680,000
Contractor's Overhead and Profit (15%)	850,000
SUBTOTAL (Construction Costs)	6,530,000
Technical Services: Legal, Administration, Surveying, Geotechnical and Engineering (20%)	1,310,000
Land Costs[b]	60,000
TOTAL	7,900,000

[a]ENR 5180.
[b]Reference 7.1.

TABLE 7.2

PRELIMINARY CONSTRUCTION COST ESTIMATE
ALTERNATIVE 2. SEQUENCING BATCH REACTORS

Item Description	Estimated Cost (1985 $)[a]
1. Headworks. Mechanical bar screen, manual bar screen, Parshall flume, grit removal tank, and septage receiving station.	160,000
2. Secondary Treatment and Nitrogen Removal. SBRs, and associated pumps, piping, valves, and controls.	1,260,00
3. Tertiary Treatment. Rapid mix and flocculation chambers, filters and spent backwash facilities, chlorination and chlorine contact tank, dechlorination.	1,660,000
4. Ancillary Facilities. Yard piping, electrical and instrumentation, control building, and laboratory.	1,130,000
5. Sludge Disposal. Sludge lagoon and drying beds.	300,000
SUBTOTAL	4,520,000
Contingency (20%)	900,000
SUBTOTAL	5,420,000
Contractor's Overhead and Profit (15%)	810,000
SUBTOTAL (Construction Costs)	6,230,000
Technical Services: Legal, Administration, Geotechnical, Surveying, and Engineering (20%)	1,250,000
Land Cost[b]	60,000
TOTAL	7,540,000

[a]ENR 5180.
[b]Reference 7.1.

TABLE 7.3

PRELIMINARY CONSTRUCTION COST ESTIMATE
ALTERNATIVE 3. PHYSICAL-CHEMICAL TREATMENT

Item Description	Estimated Cost (1985 $)[a]
1. Headworks. Mechanical bar screen, manual bar screen, Parshall flume, grit removal tank, and septage receiving station.	160,000
2. Independent Physical-Chemical Treatment. Chemical addition, coagulation and clarification, filtration, activated carbon adsorption, chlorination, sludge thickening and dewatering.	3,840,000
3. Ammonia Stripping	870,000
4. Ancillary Facilities. Piping, electrical, instrumentation, site preparation, control building and laboratory.	1,480,000
SUBTOTAL	6,350,000
Contingency (20%)	1,270,000
SUBTOTAL	7,620,000
Contractor's Overhead and Profit (15%)	1,140,000
SUBTOTAL (Construction Costs)	8,760,000
Technical Services: Legal, Administration, Geotechnical, Surveying, and Engineering (20%)	1,750,000
Land cost[b]	60,000
TOTAL	10,570,000

[a]ENR 5180.
[b]Reference 7.1.

TABLE 7.4

ALTERNATIVE TREATMENT METHODS
RANKING OF ALTERNATIVES

Item	Alternative		
	CLR	SBRs	Physical-Chem.
Personnel Required[a]	4	4	8
Annual Energy Costs ($1,000)[b]	140	140	220
Other O & M Costs ($1,000)[c]	105	105	1,880
Capital Cost ($1,000)[d]	7,900	7,540	10,570
Present Worth of Total O & M Costs ($1,000)[e]	2,870	2,870	18,780
Total Present Worth ($1,000)[e,f]	10,770	10,410	29,350

[a]Number of full-time personnel required.
[b]Costs assume $0.08 per kWh.
[c]1985 costs for personnel, materials and other O & M costs, not including energy.
[d]Itemizations of capital costs for each alternative are presented in Tables 7.1, 7.2, and 7.3.
[e]Present worth assuming 4% esclation of energy costs.
[f]Capital cost plus present worth of O & M costs, including energy.

plants now operating in the U.S. plus data on overseas plants. Although both processes have the potential to nitrify and denitrify, few U.S. plants using either process are designed for denitrification. Based on process theory, denitrification is expected to be more reliable in the SBR system due to its greater process control potential.

- The inherently modular design of the SBR, which uses several single tanks operating independently, makes this process easily amenable to future expansions required by increased flow.
- Although ammonia stripping is the most appropriate form of physical-chemical nitrogen removal for Los Osos, the physical-chemical process is far more costly than the two biological process alternatives. Both the capital and operating costs of Alternative 3 are far higher than those of Alternatives 1 and 2.

- The preliminary estimate of operation and maintenance costs are very similar for Alternatives 1 and 2. In theory, energy costs for the SBR system should be lower than those for the CLR system due to batch operation and absence if secondary clarifiers and return activated sludge pumping.
- The SBR alternative has the lowest capital cost. The capital cost of the CLR alternative is approximately five percent higher than that of the SBR alternative. This difference in capital cost between Alternatives 1 and 2 is well within the error margin of a preliminary cost estimate and therefore these alternatives must be considered nearly identical in terms of cost.

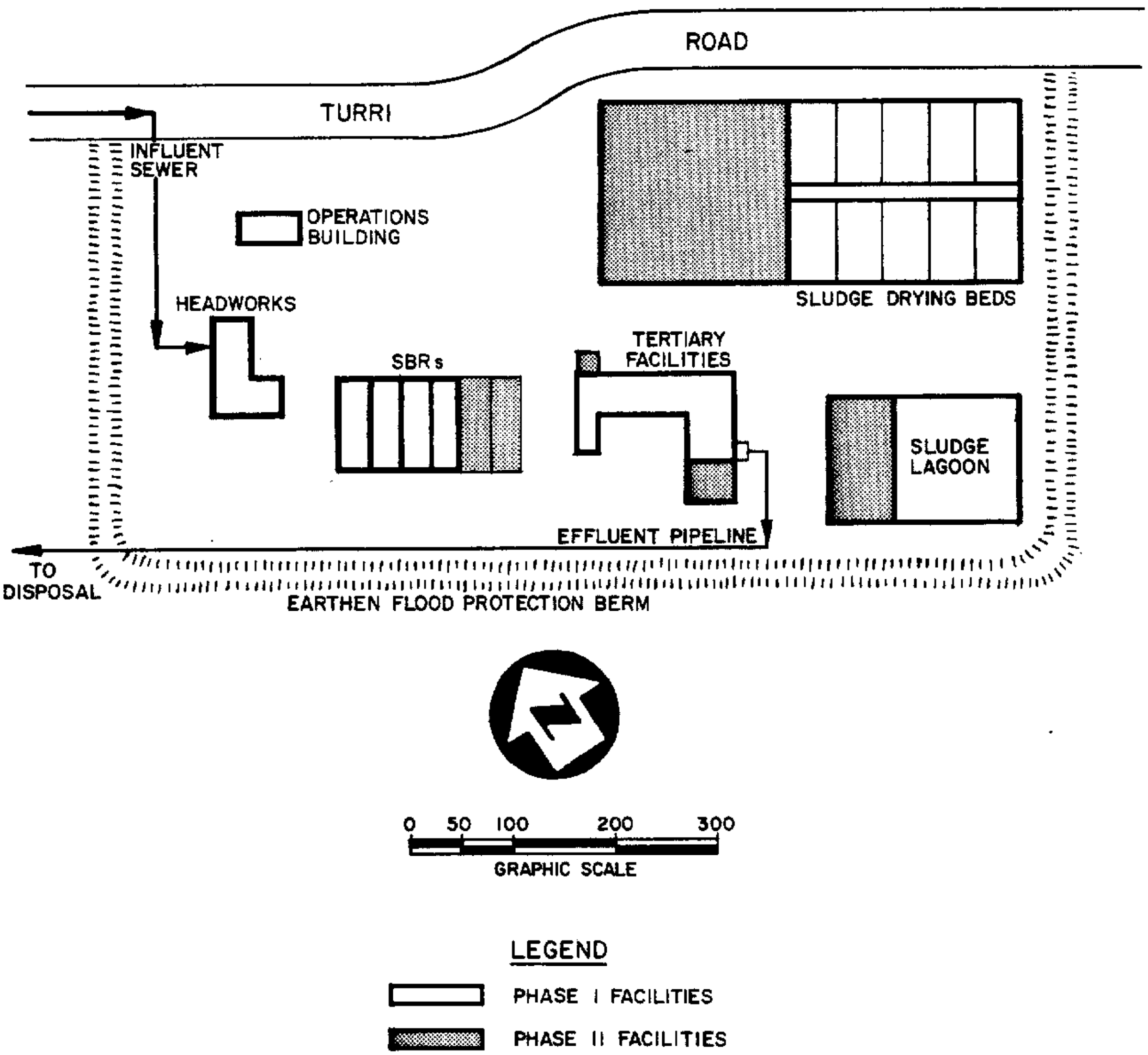
RECOMMENDATIONS

It is recommended that Sequencing Batch Reactors (Alternative 2) be selected for the Los Osos Wastewater Reclamation Facility. On the basis of cost, the present worth of Alternative 2 is only about three percent lower than that of Alternative 1. However, the SBR alternative is also preferred for its relatively low land requirements, ease of future expansion, and operational flexibility. The greater process control potential of the SBR system is particularly valuable because very stringent discharge standards are proposed for this facility.

Some caution has been exercised in selection of the SBR process due to the small number of operating SBR facilities in the U.S. Telephone and in-person interviews with operating personnel, engineers, and equipment vendors associated with currently operating SBR facilities were conducted during Phase I and should continue during Phase II.

A preliminary site plan for the recommended Los Osos Wastewater Reclamation Facility is shown on Figure 7.5.

PRELIMINARY SITE PLAN FOR LOS OSOS WASTEWATER TREATMENT FACILITIES



CHAPTER 8

CONCEPTUAL DESIGN OF DISPOSAL FACILITIES

CHAPTER 8

CONCEPTUAL DESIGN OF EFFLUENT DISPOSAL FACILITIES

GENERAL

As discussed in earlier chapters, the goal of wastewater management for the Los Osos area is to recycle the treated/renovated wastewater to the maximum extent feasible by means of groundwater recharge so as to ensure an adequate groundwater supply with acceptable quality. The groundwater recharge system will further improve water quality by filtering and adsorbing minerals, nutrients, organics, particulates, bacteria, and viruses.

Reconniassance work conducted by the Morro Group and Pacific Geoscience narrowed the potential recharge areas to two locations: (1) a 200-acre parcel above (south) Broderson Avenue, known as Site 6; and (2) Los Osos Creek, about one mile above (south) the Los Osos Valley Road Bridge. Effluent applied at Site 6 will recharge both the upper and the lower (Paso Robles) aquifer; effluent discharged to the Creek is expected to recharge the lower aquifer. For more discussion on these locations, the reader is referred to Chapter 5.

This chapter presents the conceptual design of the effluent disposal system. A transmission system would convey treated effluent from the treatment plant site on Turri Road to Site 6 and to Los Osos Creek. Infiltration/percolation ponds would be constructed at Site 6 and an outlet energy-dissipation structure would be built in Los Osos Creek. Monitoring wells would be installed to determine baseline water level and quality conditions prior to starting recharge operations and then to track water levels and certain water quality constituents with time as the system in operated.

EFFLUENT TRANSMISSION SYSTEM

General

Figure 8.1 shows the proposed layout of the effluent transmission system which will convey treated effluent from the treatment plant site to the proposed effluent disposal areas. The effluent transmission system will be comprised of an effluent pump station and effluent transmission pipelines. The effluent pump station will be provided at the treatment plant site to pump effluent to the two disposal sites through the transmission pipelines.

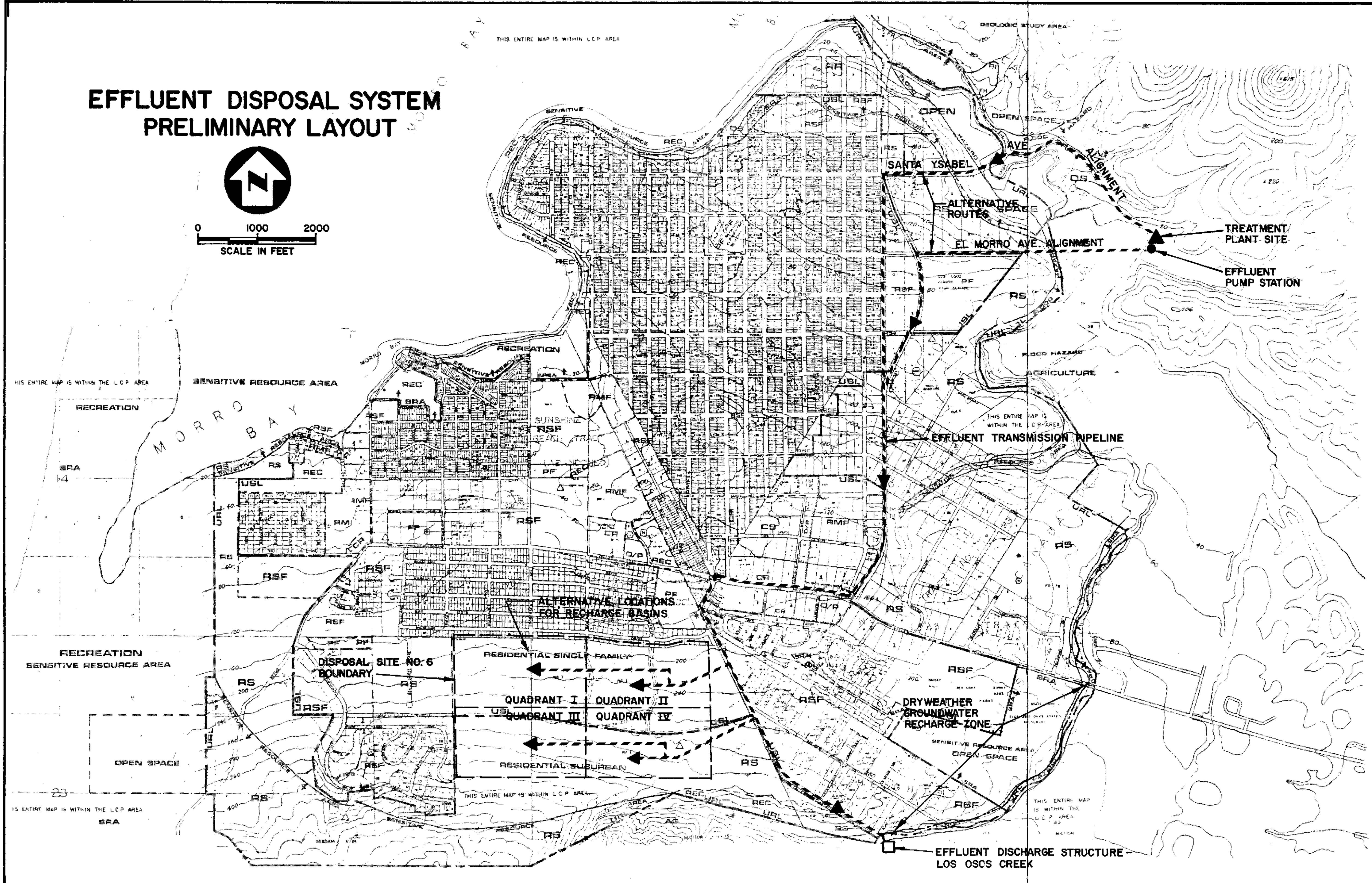
Since the Site 6 percolation ponds and the Los Osos Creek discharge point are located in the same general vicinity with respect to the treatment plant site, a single transmission pipeline will be provided to transport effluent to both locations. The pipeline will branch to discharge to the Site 6 percolation ponds and to Los Osos Creek, as shown in Figure 8.1.

Transmission Pipeline Alternative Alignments

Two alternative transmission pipeline alignments were considered to cross the Los Osos Creek wetlands between the treatment plant and the Baywood Park area. The first alternative proposes a direct crossing through the area, heading due west from the treatment plant to El Morro Avenue. The second alternative alignment would cross Los Osos Creek at the Santa Isabel Avenue bridge north of the treatment plant site. Although the El Morro Avenue route is considerably shorter and therefore less costly than the bridge crossing, other considerations such as environmental impacts may make this direct route not feasible. Figure 8.1 shows the two alternative alignments.

Currently, the precise location of the percolation ponds within Site 6 has not been defined. Although the ponds will be located within the area designated as Site 6, they will require only a portion of the total area. The final location of the percolation ponds will be determined by environmental factors (particularly Morro Bay kangaroo rat habitat), geological factors, and cost factors.

In order to quantify effluent transmission cost considerations for alternative percolation pond locations, the Site 6 area was divided into



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four quadrants as shown in Figure 8.1 Costs associated with effluent transmission to each quadrant have been developed to provide cost data for use in selection of the best location of the percolation ponds. Obviously, the higher and farther the quadrant is from the treatment plant site, the greater the cost will be to pump effluent to that quadrant. Alternative quadrant cost data will, however, establish the magnitude of such additional costs for consideration with other factors and other cost data in determining percolation pond location.

Conceptual Design Criteria

The effluent pump station will receive effluent flow from the chlorine contact chamber at the treatment plant and will pump the effluent directly into the transmission pipeline. The pumps will be vertical turbine type, driven by variable speed motors to match effluent flow rate. A separate concrete wet well will be provided for the pumps, but will have little storage capacity.

The plant SBR and other treatment processes are expected to mitigate peak flows through the plant such that the maximum diurnal peak effluent flow out of the plant should be no greater than 1.3 times the average daily influent flow. This compares with an influent diurnal peak of 2.0 times the average flow. Therefore, the dry weather design flow for the effluent pump station will be 2.1 mgd for Stage I and 3.1 mgd for ultimate development (Stage II). Considering infiltration and inflow associated with a conventional gravity sewer system, the wet weather design peak flow will be 3.0 mgd for Stage I and 4.3 mgd ultimately. (See Chapter 3 for a discussion of influent wastewater flow projections.)

The main transmission pipeline and the Site 6 transmission pipeline branch have been sized as 15-inch diameter on a preliminary basis based on the ultimate peak wet weather flow. The Los Osos Creek branch has also been sized as 15-inch diameter based on the ultimate peak dry weather flow. Transmission pipeline size selection was based on maintaining the friction head loss in the pipeline to less than 6 feet of head loss per thousand feet of pipeline at ultimate design flow.

A review of the elevations of the treatment plant site, discharge sites, and transmission pipeline profile reveals that the upper areas of Site 6 (Quadrants III and IV) and the ridge that must be crossed to reach upper Los Osos Creek establish the maximum static head that the pump station must overcome is approximately 260 feet. The static head from the treatment plant site to Site 6 Quadrants I and II is approximately 190 ft. The Stage I design total dynamic head (TDH) for the effluent pump station was calculated for each of the alternative Site 6 locations and the two alternative pipeline alignments from static head and Stage I friction losses.

Friction head loss in the transmission pipeline was calculated for discharge to the percolation ponds based on the Stage I wet weather design flow and for discharge to Los Osos Creek based on the Stage I dry weather design flow for each alternative alignment. Required horsepower based on TDH and wet and dry weather design flows was then calculated for each alternative assuming a combined pump and motor efficiency of 75 percent. The larger of the two horsepower requirements (to Los Osos Creek or to percolation ponds) determined the Stage I design horsepower for each alternative. In all cases, discharge to the percolation ponds was found to require more horsepower than discharge to Los Osos Creek due to the higher flow rate to the percolation ponds and correspondingly higher friction head losses. Effluent pump station horsepower requirements and total pipeline lengths for each alternative are summarized in Table 8.1.

The design of the treatment plant SBR tanks as currently proposed will provide several hours of influent storage at average design flow during which time no effluent discharge from the plant would occur. This storage will allow short-term shutdown of the effluent transmission system if required for repairs or normal maintenance. If it is necessary to accommodate shutdown of the effluent transmission system for a longer period than possible with SBR storage, additional storage facilities will be required if emergency discharge to Los Osos Creek at the plant site is not allowed by the RWQCB. Emergency storage could be provided by increasing the SBR tank size or by providing a lined effluent storage pond at the plant site.

TABLE 8.1

EFFLUENT TRANSMISSION SYSTEM CONCEPTUAL DESIGN CRITERIA

Alternative	Total HP[a] Required	No. of[a] Duty Pumps	HP per[a] Pump	Pipeline Diameter (inches)	Pipeline Length (feet)
El Morro Ave. Alignment					
Quadrant I	300	2	150	15	22,400
Quadrant II	290	2	150	15	20,100
Quadrant III	380	2	200	15	22,800
Quadrant IV	360	3	125	15	20,300
Santa Ysabel Ave. Alignment					
Quadrant I	350	3	125	15	25,700
Quadrant II	330	3	125	15	23,400
Quadrant III	420	3	150	15	26,100
Quadrant IV	400	2	200	15	23,600

[a]Stage I.
NOTE: HP = horsepower.

Since effluent storage requirements and emergency discharge restrictions have not yet been defined by the RWQCB, and since the cost associated with providing emergency storage will be the same for all alternative transmission systems, it is assumed that the storage provided by the proposed treatment plant design will be adequate. It is further assumed that effluent quality will be maintained within the average and peak limits set by the RWQCB. Thus storage for off-spec effluent at the treatment plant site will not be required.

Economic Analysis

Conceptual level capital cost estimates for the effluent pump station and transmission pipelines for each alternative pipeline alignment and percolation pond location are provided in Table 8.2. Energy costs associated with effluent pumping have also been estimated for each

TABLE 8.2

EFFLUENT TRANSMISSION SYSTEM ECONOMIC ANALYSIS SUMMARY
($1,000)

	El Morro Ave. Alignment				Santa Ysabel Ave. Alignment			
	Quad I	Quad II	Quad III	Quad IV	Quad I	Quad II	Quad III	Quad IV
Capital Costs								
Transmission pipelines @ $40/LF	896	804	912	812	1,028	936	1,044	944
Distribution pump station	200	200	230	227	227	227	250	230
Subtotal	1,096	1,004	1,142	1,039	1,255	1,163	1,294	1,174
Contingency (20%)	219	201	228	208	251	233	259	235
Subtotal	1,315	1,205	1,370	1,247	1,506	1,396	1,553	1,409
Contractor's overhead & profit (15%)	197	181	206	187	226	209	233	211
Total construction cost	1,512	1,386	1,576	1,434	1,732	1,605	1,786	1,620
Technical services (20%)	302	277	315	287	346	321	357	324
Total capital cost	1,814	1,663	1,891	1,721	2,078	1,926	2,143	1,944
Energy Cost	43.9	43.7	51.7	51.5	45.0	44.7	52.2	52.5
Present Worth								
Capital	1,814	1,663	1,891	1,721	2,078	1,926	2,143	1,944
Energy	662	658	779	776	679	674	787	792
Total	2,476	2,321	2,670	2,497	2,757	2,600	2,930	2,736

NOTE: See Table 8.1 for pipeline length and pump station capacities.

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alternative and are provided in terms of present worth in Table 8.2. Other O&M costs for the effluent pump station are assumed to be included in the overall treatment plant O&M cost estimate. O&M costs related to the transmission pipeline will be minimal and are assumed to be included in the collection system O&M cost estimate.

Estimated capital costs for the effluent pump station include variable speed control systems, 100 percent redundancy for the pumping units, and all required appurtenances. The pump station capital cost estimates also reflect the fact that some type of hydraulic surge control system will likely be required at the pump station to mitigate transient hydraulic surges in the transmission pipeline caused by sudden pump shut-down. Pipeline capital cost estimates include budget allowances for pipeline appurtenances such as air release and other valve assemblies.

Energy cost estimates for effluent pumping were based on initial requirements at average flow escalated at 2 percent a year to correspond to the projected average population growth during the study period. It was assumed that discharge to Los Osos Creek at the dry weather design flow would occur 50 percent of the time and that discharge to the percolation ponds at the wet weather design flow would occur 50 percent of the time. Discharge head for energy use calculations was based on static head and friction head loss for each specific discharge location. Additional head losses were added for percolation pond discharge to account for head loss in site piping. The unit cost used to estimate energy costs was $0.08 per unit, escalated at an annual rate of 4 percent to account for future increases in energy costs. The present worth of the energy costs was calculated using a discount rate of 8-3/8 percent and an evaluation period of 20 years.

As previously noted, determination of the final effluent transmission system will be a function of a number of factors other than the cost data given in Table 8.2. A review of Table 8.2 indicates that, as expected, the El Morro Ave. alignment to Quadrant II is the least costly. The El Morro Avenue alignment would provide significant cost savings over the Santa Ysabel Avenue alignment. However, with the exception of Quadrant III, the variation in cost between the Site 6 quadrants for each alignment does not appear to be substantial. As discussed elsewhere in this chapter, Quadrant IV is the least viable of the four alternative areas due to geotechnical and construction considerations. Therefore, effluent transmission system cost should not be an important consideration in determining location of the percolation ponds.

INFILTRATION/PERCOLATION PONDS

General

Groundwater recharge projects using reclaimed wastewater can be grouped into two categories; planned and unplanned, depending on whether the subsequent reuse is an unintentional byproduct of effluent discharge or is a specifically designed system following effluent discharge. EPA reports about 350 land disposal projects in California alone. About 140 of these are rapid infiltration (RI) type systems, such as is proposed for Los Osos (Reference 8.1). No distinction is made on whether these projects are planned or unplanned. On the other hand, Asano et al. report that planned groundwater recharge with treated effluent is practiced on a very limited scale. Eleven surface spreading projects and three injection projects are cited worldwide. Total quantity recharged is about 93 mgd. Nine of the 14 projects are in California (Reference 8.2). Thus, it should be pointed out that wastewater management for CSA No. 9 will be included in a limited unique group of land disposal projects practicing planned water reuse after recharge.

Recharge Basin Conceptual Design

The design of the recharge basins is dependent upon volume and rate of effluent, quality of effluent, topography, soil and hydrogeologic characteristics of the specific discharge site and of the overall area.

Wastewater flows from the project area are presented in Chapter 3. For Stage One using the average dry weather flow of 1.6 mgd for seven months and average wet weather flow of 2.3 mgd (1.6 mgd ADWF + 0.7 mgd I/I for gravity sewers) for five months yields a total annual design volume of water to be recharged of 2,090 AF. Effluent quality is discussed in Chapter 4; actual effluent quality will exceed the expected discharge standards for BOD and SS of 60 mg/L each. Even using these standards, BOD and SS loading (organic) will not control design.

The topography of Site 6 is gently sloping northward at 10 to 14 percent, depending on location within the site.

The Morro Group and Pacific Geoscience (MG/PGS) have jointly conducted reconnaissance level soils investigations and found that there is about 15 to 30 feet of windblow sand overlying the Paso Robles Formation on Site 6 (see Appendix H. MG/PGS ran soil permeability tests at seven locations and at two depths, 10 and 25 feet, at each location. Based on these tests MG/PGS recommended a hydraulic loading rate of 40 feet per day for the windblow sand unit. If spread out over a 1,000-foot east-west alignment, the wastewater would infiltrate into the Paso Robles Formation at a rate one-tenth as fast, or four feet per day. Hence, they recommended a model basin about 60 feet wide by 1,000 feet long (east-west direction) assuming 2.0 mgd of wastewater daily flow rate. This model results in an annual hydraulic loading rate of 1,626 feet per year (2,240 AF/yr ÷ 1.38 acres) (see Appendix H). EPA reports that ..."Experience in the United States with (land) treatment systems using RI has been limited to annual loading rates of about 400 feet or less" (Reference 8.1). Therefore, for the sake of conservativism, practice-to-date and the fact that very limited site data have been gathered, 400 ft/yr annual hydraulic loading was used in the conceptual design. After further field work (recommended below) is conducted, preliminary design (Phase II) refinements can be introduced and possibly the land area requirements can be reduced.

Dividing the annual volume of wastewater of 2,090 AF/yr by the annual loading rate of 400 ft/yr yields a net land area required of 5.23 acres. This will be the area of the bottom of the basins.

There will be multiple basins in the recharge system so as to allow for periods of loading, resting, drying, discing, and scarifying and maintenance. Lower basins will act as overflows from upper basins. Where the goal of the system is to maximum infiltration rate (as contrasted with nitrogen removal or nitrification objectives), EPA recommends the following ranges for loading and drying periods for infiltration with secondary efluent (Reference 8.1):

Season	Application Period	Drying Period
Summer	1-3	4-5
Winter	1-3	5-10

For Los Osos, the preliminary recommendation is for two days loading and 9 days drying. Under this arrangement, the total cycle time would be 11 days and there would be 33 cycles per year (365 ÷ 11). This cycle will result in a daily application rate of 6.06 ft/day (400 ft/yr ÷ [33 x 2 days]) which is close to the 4 ft/day recommended by MG/PGS but substantially less than the 40 ft/day MG/PGS reports for the wind-blown sand unit.

Since it was found most cost-effective from an effluent transmission point of view to locate the ponds in Quadrant II of Site 6, the conceptual design of the ponds was based on this location for two reasons: (1) the wind-blown sand unit is thickest here (25 to 35 feet); and (2) the slope is the flattest. Should the ponds be moved to another quadrant, a factor for increased costs for basin construction will be applied. Locating the basins in Quadrant IV is least desirable since the wind-blown sand unit is at its thinnest (10 to 15 ft) here.

The conceptual design is based on 12 basins, each 600 ft by 30 ft wide. Two basins would be loaded simultaneously for two days while the remaining ten are resting. No two adjacent basins would be loaded simultaneously, particularly in the north-south direction. Figure 8.2 shows preliminary basin layout and Figure 8.3 presents more detail on one basin.

Fixed piping would distribute the water to each pond. Moveable, agricultural furrow irrigation type piping would distribute the water within each basin. Six sets of distribution piping would be furnished.

Estimated Construction Costs

Table 8.3 presents the estimated construction cost of the infiltration basins. Locating the basins in Quadrant III would increase the estimated cost by approximately $150,000, because of increased earthwork owing to increased slope. Location in Quadrant IV is not recommended.

INFILTRATION BASIN LAYOUT

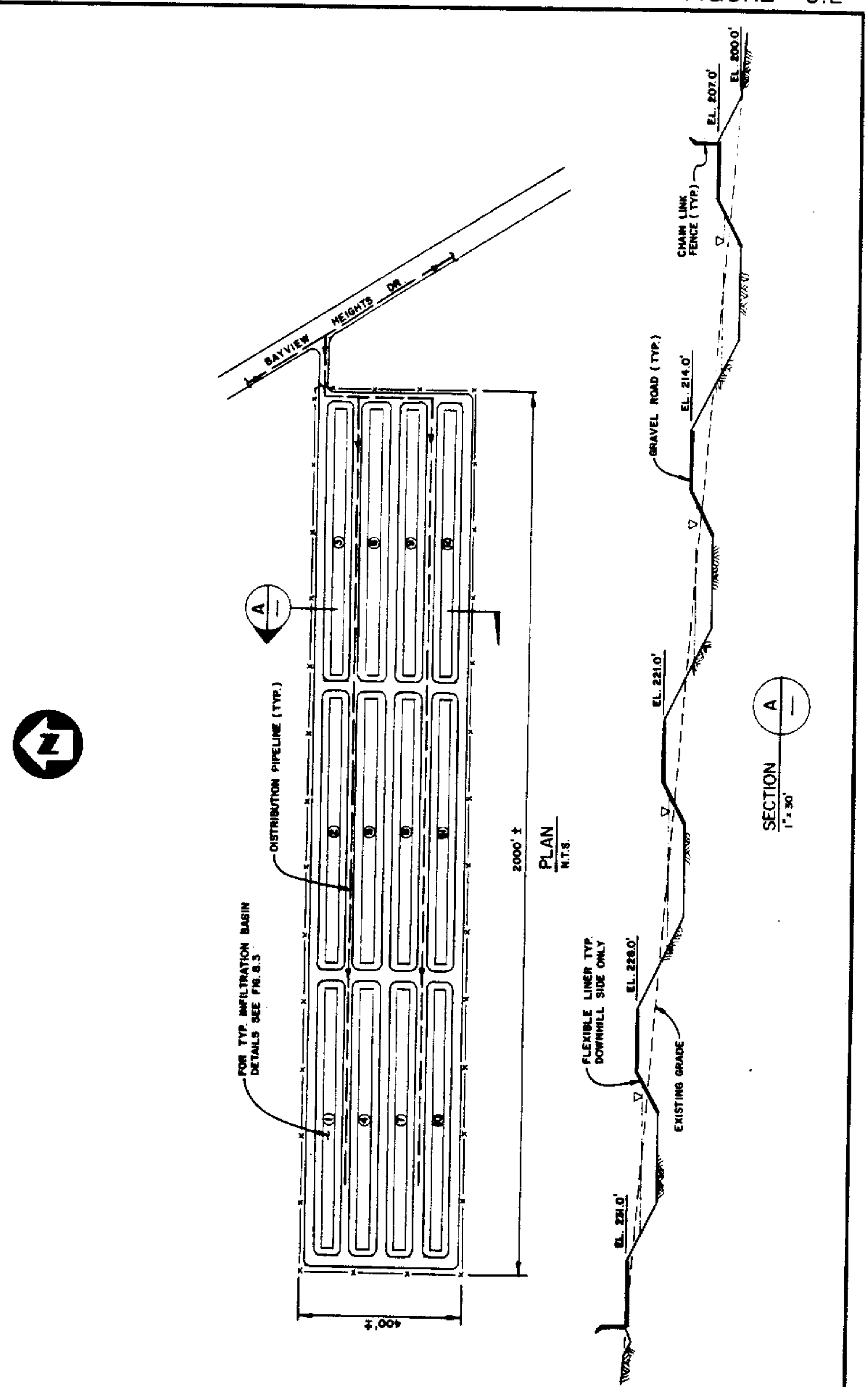


TABLE 8.3

PRELIMINARY CONSTRUCTION COST ESTIMATE FOR INFILTRATION BASINS

Item	Unit	Estimated Quantities	Unit Cost ($)	Estimated[a] Cost ($)
A. Earthwork				
Excavation	CY	54,600	3.50	191,100
Fill	CY	40,900	3.50	143,200
Removal	CY	13,700	2.00	27,400
B. Flexible Liner w/ Anchor	SF	151,00	1.50	226,500
C. Fence	LF	4,800	13.50	64,800
D. Gravel Road	SF	243,500	1.50	365,200
E. Main Feed Lines				
15" Ø pipe	LF	3,650	40.00	106,000
12" Ø pipe	LF	1,300	35.00	45,500
F. Fixed Basin Distribution Piping				
8" Ø pipe	LF	960	30.00	28,800
8" Ø flow meter	ea	24	1,000.00	24,000
8" Ø gate valve	ea	24	800.00	19,200
G. Movable Irrigation Piping				
8" Ø (aluminum)	LF	3,600	10.00	36,000
Total				1,278,000
Contingency @ 20%				256,000
Subtotal				1,534,000
Overhead and profit @ 15%				230,000
Total estimated construction cost				1,764,000

[a]ENR CCI 5180.

Land Requirements

Actual land requirements will depend on which quadrant in site 6 is selected and how much buffer zone is required. As stated above, Quadrant II was selected and it is assumed that a 200-foot buffer to the north between the lots south of Highland Drive and the basins will be adequate. Assuming a 200-foot buffer to the south and west and using Bayview Heights Drive as an easterly boundary results in a total land "take" of approximately 42 acres. Locating the basins in other parts of Site 6 will change (probably increase) land requirements. Professional appraisal advice should be sought to arrive at an appropriate budgetary value.

LOS OSOS CREEK DISCHARGE

Discharge to Los Osos Creek would be located as shown in Figure 8.1. An outlet, energy-dissipation structure is envisioned at this site. Costs for this structure would be about $20,000 to $30,000 and are considered adequately covered by the contingency for the infiltration basins as presented in Table 8.3.

ADDITIONAL STUDY REQUIRED TO VERIFY DESIGN CRITERIA FOR GROUNDWATER RECHARGE SYSTEMS

General

The analysis and design of rapid infiltration (RI) land treatment systems requires specific information on the properties of the proposed infiltration basin sites. Inadequacies in field data can lead to erroneous conclusions while excessive field data can result in unnecessarily high costs with little refinement in design. If uncertainty exists, experience indicates the adoption of a conservative approach to data gathering requirements.

Table 8.4 is a flow chart that represents the sequence of field testing for RI land treatment systems. This chart (as recommended by EPA) provides information on the type of field test data required and the appropriate sequences for collection so as to obtain sufficient data for detailed design as well as operational characteristics. As the protocol proceeds to the right (on the chart), test results may dictate reversion back to a previous step so as to verify certain results and apparent anomalies.

TABLE 8.4

FLOW CHART OF FIELD INVESTIGATIONS

	-----> Order of Testing Field Tests		
	Test Pits ----->	Bore Holes ----->	Infiltration Rate
Remarks	Usually with a back-hoe, includes inspection of existing DWR or SCS reports, etc.	Drilled or augered, includes inspection of driller's logs for local wells, water table levels.	Match the expected method of application, if possible.
Information to Obtain	Depth of profile, texture, structure, soil layers restricting percolation.	Depth to groundwater, depth to impermeable layer(s).	Expected minimum infiltration rate.
Estimates Now Possible	Need for vertical conductivity testing.	Groundwater flow direction.	Hydraulic capacity based on soil permeability (subject to drainage restrictions).
Additional Field Tests	Vertical conductivity (optional).	Horizontal conductivity.	------
Additional Estimates	Refinement of loading rates.	Mounding analysis, dispersion, need for drainage.	------
Number of Tests	Depends on size, soil uniformity, needed soil tests, type of system. Typical minimum of 3 to 5 per site.	Depends on system type (more for RI than SR), soil uniformity, site size. Typical minimum of 3 per site.	Depends on size of site, uniformity of soil. Typical minimum of 2 per site.

Source: USEPA, 1981 (Ref. 8.1).

Recommended Field Work

Bore Holes/Piezometers

Pacific Geoscience, under the direction of the Morro Group, drilled seven boreholes in sedimentary rock units underlying the Broderson Avenue site. These bores were drilled to determine both the groundwater table and the depth to the nearest impermeable layer. These depths together with certain other data are required to make mounding analyses, to design drainage facilities, and to calculate constituent (i.e., nitrogen) mass balances. In order to verify water levels at the proposed Broderson Avenue site, an additional 9 bore holes need to be drilled and these bores should have a piezometer installed in each.

Water level piezometers (one to two-inch diameter PVC) are recommended to be installed to approximately 20-foot depths in Los Osos Creek to assess its recharge capacity and characteristics. These piezometers would serve to characterize creek sediments and monitor water level fluctuations before and during discharge to the creek. Approximately five piezometers are recommended to be installed at various intervals down the creek for about a 1,000-foot reach from the proposed discharge site.

Test Pits

Shallow profile evaluation has not been performed at the Broderson Avenue site. This type of evaluation consists of digging test pits, usually with a backhoe, at each basin location. Besides exposing the soil profile for inspection and sampling, the purpose is to identify subsurface features that could develop into site limitations, or that point to potential adverse features. Conditions such as fractured, near-surface rock, hardpan layers, evidence of mottling in the profile, lenses of gravel and other anomalies should be carefully noted. At the Broderson site the wind-blown sand unit in the first 20 feet is expected to be fairly homogeneous. For proposed RI systems, this type of evaluation should extend to 3 m (10 ft) or more.

The EPA suggests that a minimum of 3 to 5 test pits be excavated per infiltration basin. Because of the uniformity of the deposits (wind-blown sands) underlying the Broderson Avenue site the suggested number of test pits per basin would result in unnecessary costs with little data enhancement. Thus, it is recommended that only 12 test pits, one per infiltration basin, be excavated in order to evaluate the shallow stratigraphic profile. Each test pit should be excavated to a depth of 10 feet.

Infiltration Tests

Proper hydrogeologic evaluation of RI systems requires the long-term acceptance rate of the entire soil surface on the proposed site for the actual wastewater effluent to be applied. Unfortunately, only the short-term acceptance rate for a number of particular areas within the overall site can be measured. The four techniques normally used to measure infiltration rates include basin flooding, cylinder infiltrometers, sprinkler infiltrometers, and air-entry permeameters. The two main categories of measurement techniques are those involving flooding (ponding over the soil area) and rainfall simulators (sprinkling infiltrometer). Flooding measurement techniques are generally preferred because of their simplicity of use, but they almost always give higher infiltration rates which in some cases can be significant (Reference 8.1). However, differences can be accommodated by adjustments to the test results.

The Morro Group performed permeability tests at two depths in their test bores on the site. These rates are reported in previous sections of this report.

At least one infiltration test using the basin flooding method should be performed on each infiltration basin for a total of 12 tests.

Hydrogeologic Assessment - Groundwater Model

In addition to infiltration rates, certain other estimates of hydraulic properties are needed to evaluate the RI operating characteristics of the proposed site and potential downgradient effects/impacts. These data include, but are not limited to, determination of the following: saturated hydraulic conductivity; infiltration capacity, to evaluate potential for ponding at the site; specific yield, important in performing groundwater mound height analyses; unsaturated hydraulic conductivity, important in determining reduced flow conductivity; and profile drainage.

Because of the short-term, non-equilibrium nature of hydraulic parameter tests and resulting estimates, a deterministic groundwater model should be constructed to simulate the actual behavior of the groundwater system in relation to the recharge site. The process of constructing a model for a groundwater management study involves data collection, data preparation, history matching, and prediction. The groundwater model can be used in all phases of the aquifer study including conceptualization and data collection, efficient design of the monitoring well array systems, detailed design of the infiltration basins, and prediction of groundwater behavior under differing operating and hydrogeologic circumstances. The ability to change the model with relative ease affords all interested parties with a useful, relatively inexpensive design tool that can save years of work as well as contingent expenditures.

The construction of a deterministic groundwater model will serve as a design tool to simulate changing conditions at the site. This initial capital expenditure could result in the saving of hundreds of thousands of dollars in construction costs, and will provide the County with a useful water management tool.

Estimated Costs

Estimated costs to conduct the additional field work necessary for design are presented in Table 8.5.

GROUNDWATER RECHARGE MONITORING PROGRAM

The objectives of groundwater monitoring are to obtain an early warning of groundwater quality degradation resulting from wastewater infiltration and to determine the effectiveness of wastewater infiltration in recharging the groundwater aquifers in the Los Osos Basin.

Monitoring wells are generally located down the topographic slope of the water table and at right angles to the center line of the groundwater flux. Since groundwater flow is three-dimensional and introduced constituents are filtered and adsorption by the deposits as they migrate, the constituents only partially penetrate aquifer systems. Hence, it is important to screen monitoring wells in that portion of an

TABLE 8.5

ESTIMATED COSTS OF ADDITIONAL HYDROGEOLOGICAL INVESTIGATIONS

Item	Estimated Cost ($)
1. Boreholes/Piezometers[a]	11,000
2. Test Pits[b]	8,000
3. Infiltration Tests[b]	6,000
4. Groundwater Model[b]	30,000
5. Evaluation, Interpretation, and Report	15,000
Total	70,000

[a]Nine at Broderson; five at Los Osos Creek.
[b]Broderson site only.

aquifer through which potential contaminants are migrating. Thus, monitoring well installation will depend on the design of the percolation basin system and will probably need to be installed at variable depths.

Los Osos Creek Site

The recommended groundwater monitoring at the proposed dry weather recharge site along Los Osos Creek requires the installation of a two-component system: one upgradient of the site and one downgradient of the site. The upgradient monitoring well array will allow for definition of native groundwater and establish background water quality.

In order to track reclaimed water constituents, such as nitrate, downgradient of the recharged zone, a monitoring well array should be situated in the residential area northwest of the site. Water level measurements near Los Osos Creek indicate that during the dry season the water table is within 10-15 feet of the ground surface. Installed wells should therefore be screened at depths no less than 10 feet and at variable depths based on hydrogeologic conditions as determined by additional site investigations such as infiltration tests, etc. Sites for additional water quality and water level monitoring stations may be needed further downgradient. Water wells from agricultural areas near the creek could serve as potential monitoring stations should the need arise.

Broderson Avenue Site

This wet weather recharge site, located west of Los Osos Creek, is underlain by 15-40 feet of wind-blown sands resting on the Paso Robles Formation. The monitoring system should observe variations in water quality, as well as serve as an indicator of groundwater mounding downgradient of the site. Mounding is an important criterion at this site due to possible surfacing of wastewater at downgradient residential areas.

In order to obtain native water quality data, a component of the monitoring system would be installed upgradient of the proposed basin system. Data from these wells will help establish background levels on water quality and the amount of water recharge from upgradient of the percolation ponds. Immediately downgradient (within 100 feet) of the recharge basins another component of the monitoring well system should be installed. The wells in this array should be installed at depths that coincide with the variable wind-blown sand/Paso Robles Formation interface. These monitoring wells should be located based on the projected geometry of effluent plumes fanning from the recharge basins.

Further downgradient (approximately 600 feet) of the recharge basins another monitoring well array would be located below the site in at an east-west street right-of-way, depending on where within Site 6 the basins are finally located. This monitoring well array would function primarily as observation points for hydraulic head measurements. Some shallow wells already in existence with historically high nitrate levels downgradient could serve to monitor groundwater quality. Deeper wells installed in the Paso Robles also exist in the general vicinity (Reference 8.3). Identification of appropriate wells will be made after selection of the recharge basin locations.

Monitoring Well Specifications

All monitoring wells will be installed in accordance with the specifications established by the RWQCB and under any other provisions and applicable ordinances of San Luis Obispo County.

Monitoring Well Surveillance

Frequency of sampling monitoring wells is based on groundwater flow rates, statistical analyses of historical variations in percolation rate changes, distance of the monitoring well from the percolation source, hydrogeologic setting of the monitoring well in comparison to that of the source, and the water table. Usually, a monthly sampling frequency is used only for establishment of initial trends, but because of the yearly limited allowable recharge time for each site a monthly sampling schedule should be initiated during the site specific recharge season. During the off-recharge season sampling can proceed on a bimonthly or quarterly schedule. Not all monitoring wells may require the same sampling schedule which will be designed with expected periodical groundwater quality and hydraulic head changes in mind.

Monitoring System Costs

Cost estimates for the monitoring system developed herein are based on historical data and analysis of construction costs and does not consider site specific conditions and availability of the required expertise and materials to complete well installations. For the purposes of this cost estimate, none of the existing wells will be assumed to be available for use in the monitoring system. The monitoring system costs are developed separately for the Los Osos Creek and Broderson recharge site.

Los Osos Creek Site. A total of 6 monitoring wells is estimated to be needed for the water level and water quality monitoring at the Los Osos Creek site. For the purposes of this cost estimate, these are assumed to average 25 feet in depth. In actuality, the wells directly adjacent to the recharge site would be at a depth of about 15 feet while the ones furthest downgradient may require depths of 35 feet.

The estimates includes the material and/or labor cost of the following:

1. Setting up and removing the drilling equipment;

2. Drilling 8-inch diameter borehole;

3. Geologic logging of the borehole;

4. Installing 2-inch diameter PVC casing, screen and fittings;

5. Gravel packing, grouting and sealing the annular space between casings and borehole; and

6. Developing the wells.

Based on the above, cost per well is estimated at $3,000. Total costs for the Los Osos Creek monitoring is thus $18,000.

Broderson Avenue Site. A total of nine wells are estimated to be needed for the monitoring system at Site 6. Estimating a depth of 25 feet per well, including the same materials and/or labor costs developed for the Los Osos Creek site, the total estimated cost for the Site 6 monitoring system would be approximately $33,000.

SUMMARY OF ESTIMATED COSTS FOR EFFLUENT DISPOSAL

A summary of the estimated cost of effluent disposal system components is provided in Table 8.6. Cost for the effluent transmission system is from Table 8.2 for the El Morro Avenue alignment to Site 6 Quadrant II. Cost for the infiltration ponds is from Table 8.3 for Quadrant II. All costs in Table 8.6 include contractor's overhead and profit and contingency. Technical and professional services fees are not included.

TABLE 8.6

SUMMARY OF ESTIMATED CONSTRUCTION COSTS FOR EFFLUENT DISPOSAL

Item	Estimated Cost[a,b] ($1,000)
Transmission	1,390
Infiltration ponds	1,760
Monitoring system	50
Total estimated construction cost	$3,200

[a]ENR CCI, 5180.
[b]Rounded to closest $10,000.

CHAPTER 9

LOCALLY SUGGESTED ALTERNATIVE SOLUTIONS

CHAPTER 9

LOCALLY SUGGESTED ALTERNATIVE SOLUTIONS

BACKGROUND

The residents in CSA 9 have taken an active interest in the solution to their wastewater collection, treatment, and disposal problem. They are particularly concerned that all cost effective and technically visible alternatives be considered. As a result of these concerns, the Board of Supervisors has directed ES to make every effort to evaluate alternatives raised by representatives of the community.

ALTERNATIVES

Ideas were solicited via interviews of interested persons by ES, and as discussed in interviews which were reported in a local paper (Reference 9.1). The principal proposer of alternative suggestions was Mr. John A. Alexander.

On 14 May 1985, a meeting was held with Mr. Alexander to discuss his alternative concepts for dealing with wastewater management in the study area. Mr. Alexander made some general suggestions and observations which are discussed below. However, he did not propose a completely defined alternative, but rather generalized and speculated on various ideas that could be considered for the area.

Many of the suggestions represented segments of a total system of collection, treatment, and disposal. Further, some of the suggestions were simply methods and materials to accomplish or implement the same design.

Since the original meeting, ES has received nothing further from Mr. Alexander, nor has ES received any concrete details concerning the suggestions offered during the 14 May 1985 meeting. However, in the past several months, there have been interviews reported in the Bear Facts newspaper wherein Mr. Alexander speculated further on science and techniques that may be applied to the project. Further, there have been several letters from Mr. Alexander to public officials concerning further observations as to ideas which should be considered, suggesting that, not to do so would be irresponsible, and implying a considerable body of empirical knowledge available to substantiate his claims.

On 8 April 1986, a follow-up meeting was held with Mr. Alexander to clarify ES' understanding of his suggestions.

The following delineates the various ideas proposed by Mr. Alexander and others for further discussion. In general, the suggestions put forth were:

1. Use a septic tank effluent (STEP) pump system in lieu of conventional collection system. This would consist of individual septic tanks with effluent being pumped from each tank in a common transmission system to a treatment facility.

2. Use a pressure sewer system consisting of a sump and grinder pump at each house and pump raw sewage to treatment in a common system.

3. Use a variable grade gravity system for transporting raw waste or septic tank effluent. This works on the principle of the pipe flowing full with higher velocities to move solids.

4. Use small diameter plastic pipe and install with a "Ditch Witch" style trencher in shallow trenches.

5. Use lightweight plastic irrigation pipe encased with reinforced concrete and lay on ground or just at the surface of the ground.

6. Leave the septic tanks and leach field lines in place. Pump the groundwater under the homes up-valley to farmland for agricultural use. If water is needed in the aquifer to hold back seawater intrusion or to supplement water supply, it can be brought down from wells at the farms up the valley in a parallel pipe. In those areas where leach fields simply cannot

be used, septic tank effluent would be pumped directly into the same line which is delivering the pumped groundwater to the farmers or by individual on-lot treatment systems. One supporting argument given for this alternative is that the difference in elevation from the community to the farms is only 16 feet. Thus, the pumping energy would be low.

7. Collect septic tank effluent by one of the low cost means described above and pump to nearby farmland for surface disposal as irrigation.

8. Instead of conventional wastewater treatment schemes, employ a new innovative chemical approach referred to by John Alexander, the inventor, as an "Electron Scrambler." This would be done on an individual house basis or as a community treatment scheme.

9. Experiment with the groundwater pump-down alternative to determine feasibility (estimated by Mr. Alexander to cost $2 million). If this approach does not prove feasible, use the transport system to try the septic tank effluent pump to irrigation scheme.

The approach of piecemeal suggestions leaves ES without a specific alternative to evaluate which is proven and will solve the problem at CSA 9 and will meet the project objectives. The ideas suggested are experimental in nature, are not based upon proven facts, and run the risk of delaying the final solution which could result in increased costs. In summary, here are the problems:

1. A number of random suggestions have been made based upon unproven principles which will require experimentation and which are well beyond the scope of this study to address.

2. Claims have been made for the cost of piping materials and methods of installation without consideration to the problems encountered and the health and safety requirements for working within a densely populated community. It is argued that the fact that such an approach has not been used in other communities is proof of the narrow-mindedness or lack of innovation by

engineers and public officials rather than the possibility that the approach is unfeasible or not workable.

3. A solution is being proposed that is dependent upon uncontrollable variables such as the weather, the market for agricultural products, the long term impacts on soil chemistry, and the whim of farmers who may not wish to continue farming land which may become valuable for other development.

4. Claims are being made for a treatment system which Mr. Alexander readily admits cannot be explained by known scientific principles and for which no independent data exist to evaluate.

5. A suggestion was that each household could have its own "Alexander device" or other individual system for which it could reclaim and reuse the water. Such an approach would depend upon each homeowner maintaining and operating his own system competently. While this concept might be acceptable in a rural area, it would be very questionable in an urban area where one person's actions affect the health and safety of his neighbor. Potential liabilities and impacts on homeowner insurance could be very significant.

QUESTIONS

There are numerous questions that these suggestions raise and which must be explored and answered if possible. These include:

1. Does pumping down the aquifer below the leach field reduce pollution from septic tanks?

2. Would the Regional Water Quality Control Board accept pump-down as a solution to groundwater pollution?

3. Would farmers take the water or could adequate land be purchased for effluent disposal and farmed by the County?

4. What happens during rainy periods?

5. Is the hydrogeology of the area suitable for a pump-down scheme?

6. What would be the area of influence of such a scheme and how many wells would be involved?

7. Is there sufficient water under the farmland to bring down to the lower end of the basin, and can it actually be developed?

8. How do you get the water, brought back, into the aquifers?

9. Can septic tank effluent be discharged on land without further treatment?

10. What is the Regional Board's position on spreading septic tank effluent?

11. What impact will sulfides in septic tank effluent have on further treatment or on irrigation of flora?

12. Does trenching down both sides of a street in the berm really reduce costs considering that twice as many utilities (water and gas) would have to be located and either cut and replaced or avoided in some other manner? Would there still have to be numerous street crossings? Further, to comply with County building codes and health department standards, there would be some constraints on where the pipes can be relative to the water supply.

13. What liabilities would the County have for surface laid piping systems or shallow systems which could be damaged, thus shutting down service or exposing the public to contamination?

14. Cost estimates for some of the alternatives have been suggested as $1 to $4 or $5 million. What does this cover (what's included in these numbers), and what percent of the total project does this represent?

15. How small a diameter pipe is feasible considering the materials which always get through a treatment system?

16. What research has been done on the "Electron Scrambler?" Where has it been used on domestic wastewaters? How does it work?

CONTINUED USE OF SEPTIC TANKS IN A SOLUTION TO THE DOMESTIC WASTE PROBLEM

In order to comment upon a solution to the CSA 9 problem which continues the use of septic tanks, a discussion of septic tank performance is necessary.

A typical household of three to four persons produces about 400 to 500 gallons of wastewater per day. This wastewater results from all the activities involving water use in the home including:

(a) laundering
(b) dishwashing
(c) garbage grinding and disposal
(d) toilet flushing
(e) showering
(f) other miscellaneous water-using activities

The use of water by persons in the home adds dissolved organics (such as sugar, urine, drinks, and juice residues) and suspended organics (such as garbage residues, feces, toilet paper, and the like). In addition to these residues, small amounts of salt and potential disease-causing organisms contaminate the waste.

In a septic tank/underground drainage system, the wastewater passes through an underground septic tank. This tank detains the waste for approximately 24 hours. During this slow passage through the tank, the suspended organic residues either settle to the bottom of the tank or are trapped in the tank as floating material.

The clarified waste then flows to the drainage system where it is infiltrated into the ground undergoing natural filtration. The natural filtration removes any harmful bacteria or virus. Thus the water percolating into the groundwater through adequate soil becomes safe from the point of view of disease. As a result of this cost-effective solution to health and aesthetics problems, many septic tank systems have been constructed throughout the world.

Problems with Septic Tank Systems

Failure of Percolation System

In the normal operation of a septic tank, there is a gradual accumulation of organic solids in the tank. If this residue (sludge) is not removed, it will ultimately fill the tank and spill over into the percolation system. The sludge produces a blinding layer in the drainage causing its failure. Failure of the drainage system then causes the wastewater to rise to the surface producing a significant health hazard to children or other persons coming into contact with the surfacing waste. Even when the sludge is pumped out of the tank, the system never works properly again due to the clogging of the drainage system which has occurred. This clogging is essentially impossible to repair.

Where tanks are pumped every two or three years, they may work satisfactorily for a lifetime. The problem is that routine pumping is often overlooked or, due to economical problems, may be put off until it is too late and overflow occurs. As a result of this, health departments across the country have very often taken very negative positions on septic tanks, attempting to thwart their construction for health-related reasons.

Problems with the Groundwater

Organic substances, both dissolved and suspended, are composed principally of the following elements: carbon, hydrogen, oxygen, nitrogen, and sulfur. A large part of the organics are anaerobically (absence of oxygen) decomposed in the environment of the septic tank. The decomposition results in the production of methane gas, carbon dioxide gas, ammonia (a soluble compound), and sulfide (a relatively soluble compound). The methane and carbon dioxide exit the system as gases through the sewer vents, harmlessly, via the roof of the residence. The ammonia and a portion of the sulfide pass out with the water phase (septic tank effluent) into the drainage system where it typically drains downward through an unsaturated zone in the soil to the groundwater below. In this unsaturated zone, there is gaseous oxygen present, thus microbes begin to grow, converting the sulfide to harmless sulfate and the ammonia to nitrate. To a limit, the deeper the unsaturated zone (depth to groundwater), the more enhanced the ammonia conversion process becomes and the more severe the nitrate problem may become.

As a parallel, the indentical problem occurs when farmers over use fertilizers containing ammonia. The residual ammonia is converted to nitrate and may appear in the groundwater. In California there are some severe problems with the latter situation.

The foregoing does not mean that all septic tank systems result in problem levels of nitrate. If the water table is deep enough, the nitrates may be retained in the soil through ion exchange. However, it is probably only a matter of time before the concentration is high enough to force the nitrates to the groundwater. If the underground water flow is adequate, the nitrates may continuously flow away resulting in a dynamic equilibrium at relatively low levels of nitrate. Unfortunately, in the study area, this is not the case and the nitrate problem is real and will continue to be real as long as septic tank systems are used for disposal of domestic waste.

TRANSITORY AGGLUTINATION THROUGH ELECTRON SCRAMBLING

Mr. John Alexander has discussed a chemical treatment system that he has developed and used in industry. While he has not directly recommended the system for CSA 9, he has discussed it in interviews with a local paper and in correspondence to various county and state officials leaving a clear implication that such a system should be considered because of the lower cost. He further suggests this system could be used on individual residences with each system reclaiming water for reuse and eliminating the need for municipal water supply.

Mr. Alexander has noted, "Because most water technicians have not yet been exposed to the science of electron scrambling or transitory agglutination, we have found it expedient to demonstrate rather than to go through the laborious task of introducing a new science" and has provided data from his research which is reproduced in Table 9.1.

Engineering-Science personnel are not familiar with a science of transitory agglutination and electron scrambling nor have we found other scientists or engineers who are acquainted with these principles. As a

TABLE 9.1

JOHN ALEXANDER RESEARCH
TREATED
MUNICIPAL SEWAGE

Test Parameter	Raw Sewage Quantity Values	Effluent Product Quantity Values	Percent Change
pH Units	10.9	7.3	
Specific Conductance, (at 25°C)	4,650	650	86.0
Total Dissolved Solids, mg/L	2,350	404	82.8
Suspended Solids, mg/L	384	1.0	99.7
BOD, 5 day at 20°C, mg/L	490	16.0	96.7
Dissolved Oxygen, mg/L(1)			
(2)	5.1	8.3	
Chemical Oxygen Demand, mg/L	741	6.0	99.2
Total Kjeldahl Nitrogen, mg/L	98.0	5.32	94.6
Ammonia Nitrogen (as N), mg/L	50.4	4.20	91.7
Organic Nitrogen, (as N), mg/L	47.0	1.12	97.6
Nitrate Nitrogen, (as N), mg/L	0.45	0.05	88.9
Sodium (as Na), mg/L	750	47.0	93.7
MBAS (Surfactants), mg/L	14.0	0.1	99.3

(1) Dissolved Oxygen value reported corresponds to amount found when sample was received. Three days elapsed between the date sample was taken and date received.

(2) Values published are from tests carried out by Quality Water Laboratories, Bellflower, California. Laboratory test #5298 on municipal sewage from Fountain Valley, California.

Reference 9.2.

result, we are not in a position to comment on the process lacking demonstrated performance on domestic wastewater. However, some observations can be made concerning the data that was submitted.

In reviewing the data presented by John Alexander on municipal sewage, it is interesting to note the apparent discrepancies in pH and dissolved solids. The data reported pH before treatment of 10.9 and after treatment of 7.3. Due to decomposition and organic acid production in sewers, sewage normally has a pH between 6 and 7. Sewage with a pH of 10.9 is very atypical.

Specific conductance and total dissolved solids measure the same thing, dissolved minerals. Specific conductivity measures the ability of the water to conduct electricity, which property is dependent on the type and concentration of the dissolved minerals. The post-treatment ratio of 404/650 = 0.62 compares well with the usual 0.64. The pre-treatment ratio of 2,350/4,650 = 0.5 is excessively low by normal standards; thus, these data are suspect.

These data indicate that 86 percent of the dissolved solids have been removed by the Alexander process. This is unheard of by a purely chemical process.

When salt is removed from water, it ordinarily appears in a brine flow or as a solid. Since there is no brine flow associated with the Alexander process, one must presume the salt will be found in the sludge or under flow as a solid substance.

The sodium concentration, a major part of the dissolved solids, is reported to go from 750 to 47 ppm. Thus the process is required to remove 83.7 percent of the sodium. There are approximately 200 inorganic salts of sodium known to science and of these 200, only four are insoluble in water and would therefore form a solid. These are:

sodium zinc uranyl acetate
sodium meta uranate
sodium trititanate
sodium metabisulfate

None of the four can exist unless the water contains large quantities of uranium, titanium, or bismith. These elements are rarely found in natural waters or sewage, certainly not in Fountain Valley, California.

Therefore ES concludes that the data presented are suspect, since they appear to violate all the laws of chemistry.

Organic constituents were reported as follows:

	Before Treatment	After Treatment
BOD	490	16
Chemical Oxygen Demand (COD)	741	6
Total Nitrogen	98	5.32
Ammonia Nitrogen	50.4	4.20
Organic Nitrogen	47	1.12
Nitrate Nitrogen	0.45	.05

Chemical Oxygen Demand (COD) is the quantity of oxygen required to fully oxidize the waste under violent laboratory conditions.

Biochemical Oxygen Demand (BOD) is the quantity of oxygen required to partially oxidize the biodegradable organics present. Therefore, BOD is some fraction of COD. In the raw waste, the fraction reported is 490/741 = 66 percent, which is typical for wastewaters. In the treated effluent, the BOD is 16/6 = 2.7 times the COD. This is very unusual but it might be explained if the BOD test were run under nitrifying conditions. Ammonia nitrogen is not measured under the conditions of the COD test, thus the biological oxidation of ammonia to nitrite and nitrate could explain the data. In any case, the removals are truly remarkable if no error is present.

In general the data presented are very suspect. Since this information is not corroborated, it cannot be accepted as evidence that the proposed treatment system works as suggested.

Another study of the Alexander system was performed by the County on septic tank effluent at Black Lake. A memorandum reporting the results of those tests is presented in Appendix I. The concentrations of various constituents in the effluent do not come anywhere near meeting the effluent quality requirements for wastewater treatment. In fact, they could not be classified as even the result of primary treatment. While nitrates were reduced, the concentrations of ammonia nitrogen NH_3-N were substantially increased. This ammonia can be converted to nitrate in the environment through the same mechanism that the nitrates which are now finding their way into the groundwater are produced. These results are inconsistent with the results in Table 9.1. For a treatment process to be suitable for treating wastewaters, it must be reliable and produce reproducible results.

The results obtained thus far for the Alexander system on domestic wastewaters are at best inconclusive and strongly suggest that it may not be applicable for CSA 9, certainly not without considerable research and demonstration as to its reliability both as a process and as a piece of equipment.

In order to insure that the system has been thoroughly and fairly investigated, ES has requested a demonstration system from John Alexander and Associates which will be operated on domestic sewage at a plant in the Bay area. A report of the performance of the system will be submitted at a later time.

NON-CONVENTIONAL ALTERNATIVES

Those alternatives which considered such ideas as septic tank effluent pumping (STEP), variable grade sewers (VGS), and pressure sewers are evaluated elsewhere in this report. They represent conventional alternatives because there exists a state-of-the-art and actual operating experience by which to evaluate the alternatives. The purpose of this section is to evaluate non-conventional alternatives reflecting ideas which are put forth by the public where there is insufficient experience, examples, or technical data by which to judge them.

The ideas suggested fall into two categories. One category is System Alternatives which are intended to solve the total problem and the other category is Non-System Alternatives which are ideas to reduce the cost of some component of the system.

Based upon the discussion and ideas set forth in the various media discussed previously, the following alternatives have been identified.

System Alternatives

SA-1 - Continue use of septic tanks. Pump the groundwater down and transport up the valley for farm irrigation. If supplemental water is needed for domestic supply or as a sea water intrusion barrier, it will be taken from under the up-valley farms and transported back to the residential areas. The purpose of this alternative is to separate the groundwater table from the leach field to reduce nitrates because it has been alleged that nitrates will not appear (are removed or not produced) in the groundwater when there is sufficient separation between the water table and the leach field. It was proposed that for those septic tank systems that remain in a flooded area, the septic tank effluent will either be discharged into the system taking the groundwater to the farms, or individual treatment systems such as the Alexander system would be used before combining with the groundwater.

SA-2 - Continue use of septic tanks. Pump the groundwater, remove the nitrates and return to the domestic supply system. Flooded septic systems which will not be improved by this approach would be treated by a separate system.

SA-3 - Continue use of septic tanks. Collect effluent via a STEP system and transport it up-valley for disposal on farm land.

Non-System Alternatives

NSA-1 - Use plastic, small diameter pipe and install with a "Ditch Witch."

NSA-2 - Use plastic or thin wall irrigation pipe encased in reinforced concrete and placed on the surface or just below the surface.

NSA-3 - Use individual home treatment systems or cluster systems. The effluent would either be used locally by each home or collected for common disposal.

DISCUSSION OF NON-CONVENTIONAL ALTERNATIVES

SA-1

The facilities required for this alternative include:

1. Connection of existing wells and installation of new wells to insure uniform drawdown of water table.

2. Collection piping and transport main to irrigation system.

3. Irrigation distribution system.

4. Wet weather storage reservoir of approximately three months capacity.

5. Purchase of land and development of a farming program to use the water. (Preliminary discussions have indicated that current farmers are not interested in such a program.)

6. Up valley collection wells.

7. Collection piping and transport main to community distribution system.

8. Groundwater injection system for sea water intrusion barrier.

9. Installation of some type of system to manage wastewaters from areas where this pumping scheme will not relieve septic tank flooding problems. (It has been suggested that these waste waters simply be blended into the groundwater being pumped. This would contaminate water that is otherwise safe from disease hazards. This would not be acceptable to farmers or any regulatory agency due to the serious health-related risk.)

There are a number of problems raised by this alternative that cannot be solved within the scope of this study. These are:

1. Can the groundwater basin be lowered and managed within reasonable limits? How much more flow would have to be pumped than is contributed by septic tanks?

2. How far must the water tabled be lowered to affect a nitrate reduction or will it continue to remain contaminated?

3. Is there adequate up-valley groundwater to offset what is being pumped and lost to irrigation?

4. The water being pumped to the up valley fields contains high nitrates, some fraction of which will be removed during the growing season. During the non-growing season they will be returned to the groundwater producing a gradual buildup in nitrates in the upstream groundwater. Ultimately, since this is the new source of drinking water, the old problem of nitrates will return, perhaps at a somewhat reduced level.

5. The basin plan will have to be modified to permit nitrates in both the downstream and the upstream groundwaters.

6. There remains the problem of treating the wastewaters from flooded septic systems. To date there is not a proven, reliable, cost effective system which could be used on individual households in the CSA 9 project area.

While there can be little doubt that a pump-down and irrigation scheme would cost less than complete collection, treatment, and disposal, this proposal would result in an incomplete solution to the total problem, would be subject to variables over which the operators have no control, would rely on physical/chemical environmental principles for which there is disagreement between competent experts, and would be dependent upon unknowns which no amount of study might resolve. It does not appear that this alternative is feasible.

SA-2

The state-of-the-art for nitrate removal from groundwaters is still in its infancy. The various potential alternatives are very expensive and have not been proven for domestic supply.

The problem of the flooded septic systems which will not be improved by this scheme remain a problem whose solutions are subject to the same limitations as discussed under Alternative SA-1. In conclusion, this alternative does not appear feasible.

SA-3

Because of potential health and odor problems septic tank effluent would have to be treated. Since a community-wide collection system, a treatment system, a septic tank maintenance program, a farm system, and a wet weather management system would all be required, there would be no cost advantage over the conventional system which will be returning water to the groundwater basin for domestic reuse. In conclusion, this alternative does not appear feasible.

NSA-1

This idea may have some limited application where pressure pipes are used of small diameter and where there are no interferences. This would have to be judged on a case-by-case basis during design. However, it is unlikely that there will be major cost reductions realized.

NSA-2

Pipes laid on the surface are simply unacceptable from a safety and public health standpoint. Pipes are generally buried a minimum of 3 feet to protect them from impact loads of truck, cars, heavy equipment, and other activities found in a community. Such burial also protects them from other activities such as grading for street improvements, structural foundations, landscaping, and other digging activities. Actual depths are determined by utilities and drainage features, desire to be able to drain the pipes and whether they are pressure pipes or gravity flow.

Notwithstanding that the pipes laid just at the surface would be encased in reinforced concrete, they would still potentially be subjected to forces (anticipated and unanticipated) which could rupture the pipes creating health hazards. Further, it is difficult to conceive a community with ribbons of concrete at or near the surface running down all the streets from each property.

NSA-3

Individual on-lot treatment and reuse systems are not presently feasible for high density areas. Not only has the technology not been suitably developed for such a scheme, but the potential liability in

CHAPTER 10

THE RECOMMENDED PLAN

CHAPTER 10

THE RECOMMENDED PLAN

DESCRIPTION OF FACILITIES

The recommended wastewater management plan for County Services Area No. 9, serving Los Osos, Baywood Park, and Cuesta-By-the-Sea involves a conventional gravity sewage collection system, tertiary treatment plant, and effluent reuse by groundwater recharge to both the upper and the lower aquifers.

The collection system will consist of approximately 46 miles of gravity collection, 3 miles of force mains, a 1-1/2 mile gravity interceptor, 5 pump stations, and 57 miles of service laterals. Small pockets of pressure sewers may be used to avoid excessive cuts in certain low areas. The decision of where to use these will be made during the design phase.

The treatment plant will be located approximately one mile east of South Bay Boulevard off Turri Road. The plant will consist of a headworks containing screening, grit removal, and septage receiving. Secondary biological treatment for removal of biochemical oxygen demanding substances and nitrogen will be by sequencing batch reactors (SBRs). Separation of the bio-mass from the wastewater is accomplished in the reactor basins; separate clarifiers are not provided. After the SBRs, the wastewater will be flocculated with alum and polymers, filtered, disinfected by chlorination, dechlorinated and then pumped to the reuse/recharge areas. Sludges removed by the processes will be dewatered on sand drying beds. The dried cake will be hauled by truck to a local landfill.

An effluent pump station and approximately four miles of force main will transport treated wastewater to the reuse areas where it will be used to recharge both the wind-blown sand unit formation ("upper aquifer") and the Paso Robles Formation ("lower aquifer"). Recharge of the upper and lower aquifer will be accomplished by spreading effluent in rapid infiltration basins located south of Highland Drive and west of Bayview Heights Drive. The basins are designed to work year-round. However, in summer when there is no flow in Los Osos Creek, treated effluent will be discharged to the Creek about one mile above Los Valley Road bridge.

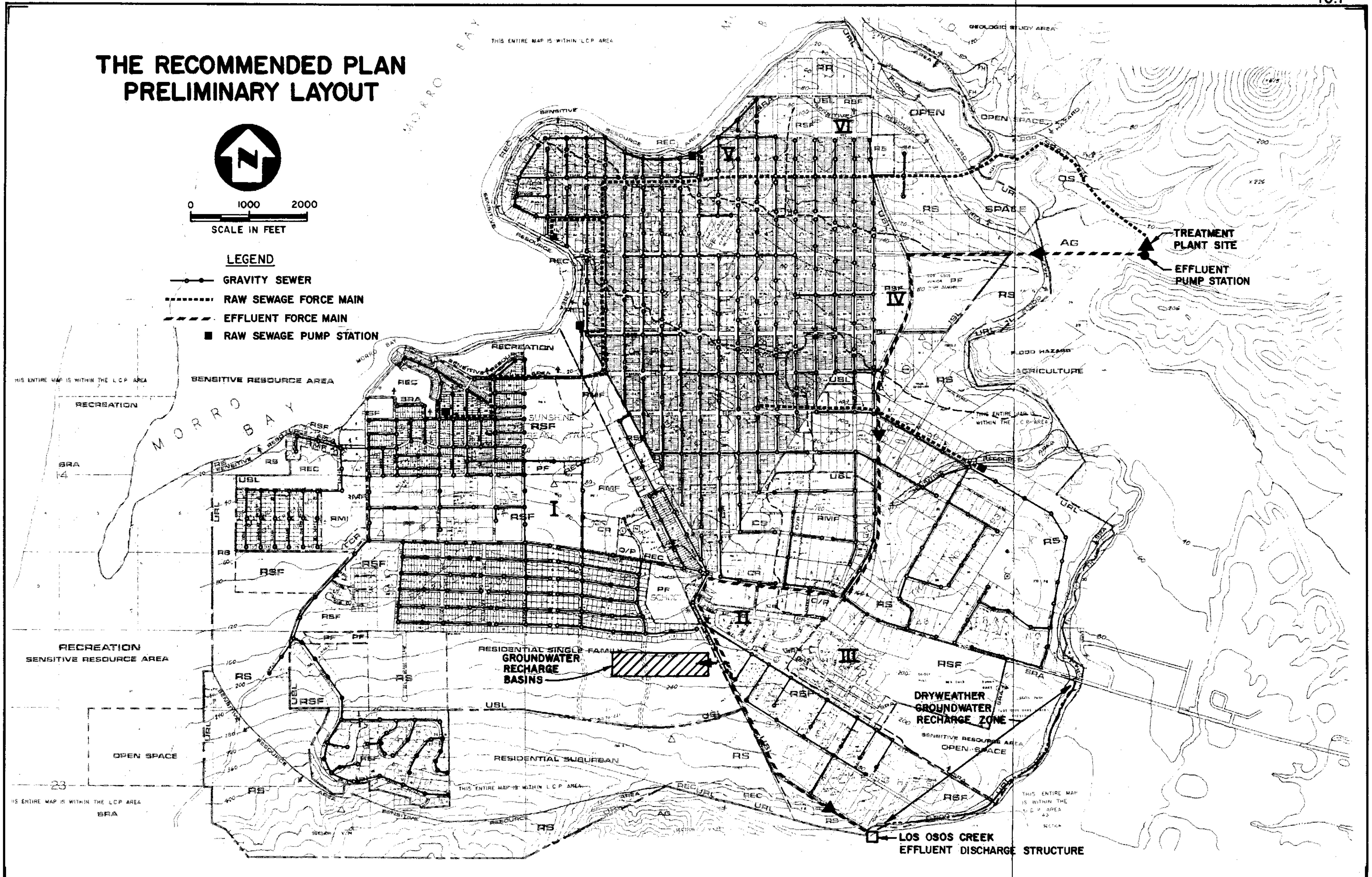
An overall site plan of The Recommended Plan in presented in Figure 10.1.

SUMMARY OF ESTIMATED COSTS

For details on the cost estimates, the reader is referred to Chapters 6, 7, and 8. Summaries of the estimated capital and operations costs are presented herein. Since ES has no control over the cost of labor, materials or equipment, or the general inflation of prices, or over contractors' methods of determining prices, the estimates of construction cost provided herein have been prepared on the basis of experience and judgment of an engineering professional. But ES does not and cannot guarantee that proposals for construction will not vary from opinions of probable cost prepared by ES. Moreover, the cost estimates provided herein are conceptual in nature and were developed for use in evaluating and selecting alternative systems. Pursuant to the ES-County Contract, "...These cost estimates are not intended to be adequate for detailed financial planning."

Capital Costs

The estimated capital costs for the Recommended Plan are summarized in Table 10.1. Estimates for engineering, administrative, legal, and financing will need to be added along with inflation to the midpoint of construction as well as interest during construction to obtain a total project cost estimate.



THE RECOMMENDED PLAN
PRELIMINARY LAYOUT
0 1000 2000
SCALE IN FEET
LEGEND
GRAVITY SEWER
RAW SEWAGE FORCE MAIN
EFFLUENT FORCE MAIN
RAW SEWAGE PUMP STATION
TREATMENT PLANT SITE
EFFLUENT PUMP STATION
RESIDENTIAL SINGLE FAMILY
GROUNDWATER RECHARGE BASINS
DRYWEATHER GROUNDWATER RECHARGE ZONE
LOS OSOS CREEK EFFLUENT DISCHARGE STRUCTURE
MORRO BAY
SENSITIVE RESOURCE AREA
RECREATION
OPEN SPACE
RESIDENTIAL SUBURBAN
AGRICULTURE
FLOOD HAZARD
GEOLOGIC STUDY AREA
THIS ENTIRE MAP IS WITHIN L.C.P. AREA

TABLE 10.1

SUMMARY OF ESTIMATED CAPITAL COST OF THE RECOMMENDED PLAN

Item	Estimated Cost[a] ($ million)
1. Collection System	20.6
2. Treatment Plant	7.2
3. Effluent Transmission/Disposal	3.2
Total estimated construction cost[a,b]	31.0
Construction cost update[c]	32.2
Estimated technical services[d] (20%)	6.4
Land	0.5
Total estimated capital cost	39.1

[a]Based on ENR CCI 5180 May 1985 (average LA & SFO).
[b]Includes allowances for 20% contingency and 15% contractors' overhead and profit.
[c]ENR CCI update to April 1986, ENR CCI 5377 (average LA-SFO).
[d]Engineering design, CM, easement acquisition, surveying, geotechnical, hydrogeological (see Table 8.5), legal, financial, and administrative services. Does not include costs for bond counsel, bond sales commissions, interest during construction and the like.

TABLE 10.2

SUMMARY OF ESTIMATED FIRST YEAR O&M COST OF THE RECOMMENDED PLAN

Item	Estimated Costs[a] ($1,000/year)
Collection	165
Treatment	245
Disposal	
Effluent	60
Sludge	10
Administrative	20
Total estimated O&M costs	500

[a]April 1986.

CHAPTER 11

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

CHAPTER 11

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

Chapter 1 - Introduction

The RWQCB has concluded that the quality of groundwater in the Los Osos/Baywood Park area of San Luis Obispo County is being degraded by excessive nitrate and bacteria concentrations emanating from septic tank/leach field systems currently used for wastewater disposal in the area. In order to mitigate this decline in groundwater quality, the RWQCB has adopted an amendment to the Basin Plan which prohibits the use of septic tank/leach field systems in the area as of 1 November 1988.

The purpose of this study is to define the most cost-effective and technically feasible methods for collection, treatment and disposal of sewage from the area in accordance with the requirements of the amended Basin Plan and the RWQCB. The results of this study will be used as the basis for subsequent preliminary design and financial planning for the sewerage project.

The objective of the sewerage project is to provide a system which will perform reliably and which will impose the minimum level of financial burden to users while protecting and preserving the quality of groundwater in the area. An additional objective of the project is to maximize recharge of the groundwater basin.

Chapter 2 - Study Area Characteristics

The study area includes the westerly draining half of Los Osos Valley and Clark Valley in San Luis Obispo. The service area for the project is CSA No. 9, referred to as Los Osos/Baywood Park. The topography of the service area is highly variable due to the local convergence of the Irish Hills, Los Osos Valley, Pacific Ocean, Morro Bay, and Los Osos Creek. After a period of high population growth in the 1970s, the growth rate in this primarily residential community appears to have begun to stabilize. The current population of the area is estimated to be 13,100. The County estimates that the ultimate population of the service area will be 28,200. The Stage I design population project for the year 2000 is 18,700. Stage II expansion of the facilities to accommodate the ultimate population will occur when the Stage I design population is reached.

Surface water features of the Los Osos hydrologic basin include Los Osos Creek and its tributaries, Eto Creek, and a few small lakes and impoundments. Groundwater occurs in an upper aquifer zone that includes 100-200 feet of old dune sand deposits and in a lower aquifer that includes most of the Paso Robles Formation. Wastewater disposal within the service area is presently accomplished with individual, on-site disposal systems (primarily septic tanks and leach fields) and several community septic tank-leach field systems.

Chapter 3 - Water Supply and Wastewater Characteristics

Groundwater from wells supplies all domestic and agricultural water requirements in the study area. In the service area, chlorinated well water is distributed by municipal water purveyors. No water is currently imported to the area.

Wastewater flow projections are based on population projections from Chapter 2 and an 85 gpcd unit flow rate. Average dry weather flow for Stage I design is 1.6 mgd and is 2.4 mgd for ultimate development. A dry weather peak factor of 2 was assumed to account for diurnal flow fluctuations. Wet weather infiltration and inflow projections were also calculated based on alternate sewer systems (see Table 3.2). Wastewater characteristics are expected to be typical of domestic wastewater. Septic tank effluent which would flow to the treatment plant from some of the alternative sewer systems will have low dissolved oxygen and decreased suspended solids and BOD.

Chapter 4 - Wastewater Discharge and Treatment Requirements

Discharge requirements are based on effluent recharge of potable groundwater supplies. Preliminary requirements have been set by the RWQCB and include Title 22 requirements for groundwater recharge (including coliform bacteria 7 day average of 2.2 MPN/ml) and total nitrogen of 5 mg/l as N. BOD, TSS, DO and turbidity requirements for discharge to Los Osos Creek are more restrictive than discharge to percolation ponds. Discharge to Los Osos Creek will not be allowed when surface water continuity exists between the Creek and Morro Bay. Treatment plant effluent quality criteria are based on discharge to Los Osos Creek, which has the most restrictive discharge requirements of the planned disposal alternatives. To achieve the adopted effluent quality criteria, screening, grit removal, secondary treatment, nitrogen removal, and other tertiary treatment processes will be required.

Chapter 5

Wastewater disposal by groundwater recharge requires a hydrogeologically suitable site which will allow percolation of effluent into the groundwater basin. The Los Osos groundwater basin consists of two aquifer zones above a basement complex. Selection of alternative sites for groundwater recharge was initially based on the assumption that the two aquifer zones are discrete, although some interconnection is believed to exist. Since the primary source of municipal water for the area is from the lower aquifer zone, recharge to this zone is preferrable to recharge to the upper aquifer zone.

The major source of recharge to the groundwater in Los Osos Basin is through precipitation. Wastewater from septic tank leach fields is currently an additional significant source of recharge to the upper aquifer zone. The geology of the upper aquifer zone is comprised primarily of highly permeable sand dune deposits. Permeability of the series of marine sediments which comprise the lower aquifer zone (Paso Robles Formation) is also relatively high, but less than the upper zone.

After the investigation of five potential sites for recharge of treated effluent to the lower aquifer, it was determined that no feasible sites for direct lower aquifer recharge exist. Recharge of the upper aquifer (and potential indirect recharge of the lower aquifer) is now planned via a combination of discharge to Los Osos Creek during dry weather and to percolation ponds during wet weather. Discharge to Los Osos Creek may result in direct recharge of the lower aquifer zone. The percolation ponds will be located at Site 6 south of Los Osos and near Broderson Road near the southern and upper edge of the upper aquifer zone (see Figure 5.1 for Site 6 location).

Chapter 6 - Development and Evaluation of Alternative Collection Systems

The capital cost of the sewage collection system for the project represents roughly 80 percent of the initial cost estimate for the total project. Alternative collection systems have been evaluated to determine if use of these systems in lieu of the conventional gravity sewer system originally proposed for the project can substantially reduce the cost of the sewer system, and thus the cost of the overall project.

Five types of sewage collection systems were evaluated. The first was a conventional gravity sewer system, which was used to set comparison standards of cost and performance. The alternative sewer system types which were evaluated consisted of pressure sewers with STEP pumps, pressure sewers with grinder pumps, and variable grade gravity sewers.

In general, the capital costs of alternative systems are lower than for a conventional gravity sewer system but O&M costs of alternative systems are much higher than conventional systems. Alternative systems typically have additional nuisances not usually associated with conventional systems such as potential for unforeseeable maintenance costs, septic tank maintenance, pump systems located in users' yards, odors from vented lines containing septic tank effluent, and so forth. In the appropriate situation, however, the cost savings from the use of alternative systems has been found in some instances to outweigh any associated uncertainties and nuisances.

Based on an initial evaluation, conceptual level designs were prepared for the conventional system, several alternative pressure sewer systems with STEP pump units, and a combination system of conventional gravity sewers and pressure sewers. Although STEP pump units require a septic tank, they were selected over grinder pumps due to lower capital cost, maintenance and energy consumption and because the septic tank provides storage capacity if the pump fails. It was determined that variable grade gravity sewers were not technically feasible for this large scale application and that the cost of such a system would be comparable to the other sewer systems. The objective of the combination system alternative was to utilize pressure sewers in those areas where they would have the greatest impact in terms of cost reduction.

A present worth economic evaluation of these final alternative systems was performed based on capital and annual O&M cost estimates derived from the conceptual designs. Although a pressure sewer system with one STEP unit serving two users was found to have the least present worth cost, it was determined that the cost savings afforded by this pressure sewer system did not justify the associated increase in potential for unforseeable additional O&M costs. The combination system also did not yield the anticipated level of cost savings. Thus, it was determined that the alternative sewer systems could not provide cost savings of the magnitude which could justify their implementation in this particular application. A conventional gravity sewer system with limited use of pressure sewers as appropriate in the most troublesome areas is therefore the recommended sewage collection system for this project.

Chapter 7 - Evaluation of Alternative Treatment Systems

The treatment facilities must provide tertiary treatment including nitrogen removal to meet the anticipated RWQCB discharge requirements. Three alternative treatment processes which would accomplish nitrogen removal were evaluated. The first two were biological processes; continuous loop reactor and sequencing batch reactors (SBRs). Although both these processes have the potential to nitrify and denitrify, few U.S. plants using either are designed for denitrification. The third alternative consisted of physical-chemical treatment processes for wastewater treatment including nitrogen removal. The present worth costs for the two biological processes were approximately the same and were both about 25 percent less than the physical-chemical alternative.

Sequencing batch reactors are the recommended treatment process for this project. The SBR alternative present worth was only slightly less than the continuous loop reactor. The SBR alternative is preferred for its relatively low land area requirements, ease of future expansion and greater operational flexibility. The potential for greater process control for the SBR is particularly valuable in this case because of the stringent discharge requirements anticipated for this facility. Although SBR technology is relatively new in the U.S., many SBR plants are operating successfully in other countries and many are planned for construction in the U.S.

Chapter 8 - Conceptual Design of Effluent Disposal Facilities

Disposal of effluent from the wastewater treatment facilities is proposed to be accomplished by reuse for groundwater recharge. An effluent transmission system would convey treated wastewater from the treatment plant site to remote infiltration/percolation ponds located in the north-east quadrant of Site 6 south of Highland Drive and to Los Osos Creek.

The transmission system would consist of a pump station at the plant site, and a transmission pipe to convey effluent to the disposal sites. Infiltration/percolation ponds would be constructed at Site 6 and an outlet energy-dissipation structure would be constructed at the Los Osos Creek discharge point. Ponds would be designed as a rapid infiltration type recharge system. Multiple basins will be provided in the system to accommodate individual pond wetting and drying cycles. The basins will require approximately 42 gross acres and will be constructed in the north-east quadrant of Site 6 based on transmission system costs and geotechnical considerations.

Prior to final design of the infiltration/percolation ponds, additional study of the geology and groundwater characteristics of the proposed pond site will be required. This study should include bore holes to verify groundwater levels, test pits for geological investigation, infiltration rates, and deterministic groundwater modelling. Budgetary estimate for this work is $70,000.

Groundwater quality monitoring would be required as part of the effluent disposal system to ensure that effluent recharge is not degrading overall groundwater quality. Monitoring wells would be installed downgradient of the two recharge sites. Monitoring of groundwater mounding would also be accomplished with these wells, particularly at Site 6 to avoid surfacing of recharge in downgradient residential areas.

The estimated cost for the effluent disposal system including transmission system, infiltration/percolation ponds and a groundwater monitoring system is $3.2 million.

Chapter 9 - Locally Suggested Alternative Solutions

The residents in CSA 9 have taken an active interest in the solution to their wastewater management problem. They are particularly concerned that all cost-effective and technically visible alternatives be considered. As a result of these concerns, the Board of Supervisors has directed ES to make every effort to evaluate alternatives raised by representatives of the community. Ideas were solicited via interviews of interested persons by ES. Alternative concepts were also reported in a local paper, The Bear Facts. The principal proposer of alternative suggestions was Mr. John A. Alexander. Meetings were held with Mr. Alexander to synthesize a body of rather general and speculative ideas, concepts, and alternatives.

The ideas suggested fall into two categories. One category is System Alternatives which are intended to solve the total problem and the other category is Non-System Alternatives which are ideas to reduce the cost of some component of the system. Based upon the discussion and ideas set forth in the various media discussed previously, the following alternatives were identified.

System Alternatives

SA-1 - Continue use of septic tanks. Pump the groundwater down and transport it up the valley for farm irrigation. Furnish supplemental domestic water from wells on up-valley farms.

SA-2 - Continue use of septic tanks. Pump the groundwater, remove the nitrates, and return to the domestic supply system.

SA-3 - Continue use of septic tanks. Collect effluent and transport it up-valley for disposal on farm land.

Non-System Alternatives

NSA-1 - Use plastic, small diameter pipe and install with a "Ditch Witch."

NSA-2 - Use plastic or thin wall irrigation pipe encased in reinforced concrete and placed on the surface or just below the surface.

NSA-3 - Use individual home treatment systems or cluster system employing a treatment system developed by John Alexander and called "transitory agglutination through electron scrambling."

Several problems and shortcomings were identified with each of the above alternatives, including the following:

- Regulatory - Degradation of groundwater quality would continue under SA-1 and SA-2, necessitating a change to the Basin Plan adopted as law by the RWQCB.
- Liability - Surface laid pipe could be broken resulting in public health problems and interruption of service.
- Operating - Under NSA-3, each home owner would be responsible for operation and maintenance of his own system. Ignoring failures could cause public health problems.
- Water Supply/Use - Farmers do not appear eager or willing to use the water. The potential "demand" is less than the supply. Using more water for irrigation creates a new use, thus not conserving water. Hydrogeology of up-valley aquifers is unknown.
- Process Engineering - Operating data from John Alexander's electron scrambler are suspect and appear to defy the laws of chemistry.

While some elements of the non-conventional alternatives appear attractive at first glance, unfortunately they either do not provide complete solutions or are dependent upon technical conjecture.

Chapter 10 - The Recommended Plan

The Recommended Plan for CSA No. 9, serving Los Osos, Baywood, and Cuesta-by-the-Sea, includes a gravity collection system, a tertiary treatment plant, including nitrogen removal and filtration, and effluent disposal/reuse employing groundwater recharge by rapid infiltration basins. The total estimated project cost is $39.1 million in April 1986 dollars. First year operating cost is $500,000. Eight full-time staff persons plus a secretary/clerk will be required to operate and maintain the system. Additional administrative support for billing and management would be required from the County.

CONCLUSIONS

As a result of the investigations, alternative analyses, and conceptual design activities conducted during the planning process, certain conclusions regarding wastewater management for San Luis Obispo County Service Area No. 9 can be drawn:

1. The Study Area is becoming water-short owing to a projected overdraft on the groundwater basin.

2. Nitrate levels in the groundwater are rising owing to continued operation and proliferation of septic tank leach field disposal systems.

3. Based on Conclusions 1 and 2 above, an appropriate wastewater management goal is to recharge the local aquifers to the maximum extent feasible. The County has gone on record stating this goal.

4. By the year 2000, the County Planning Department projects that the population of the Study Area will grow from the present 13,100 persons to 18,700, or 43 percent. Ultimate population is projected at 28,200.

5. Using the population projections from above, the Stage One (year 2000) treatment system will be designed for an average dry weather flow capacity of 1.6 mgd. Allowances for infiltration and inflow will be 0.7 mgd during wet weather.

6. Two locations for groundwater recharge appear feasible. One for rapid infiltration by spreading in large basins is just south of Highland Drive between Cabrillo Estates and Bay View Heights. Infiltration here will recharge primarily the upper aquifer with additional penetration into the low aquifer. The other site is in Los Osos Creek, for the one mile reach above Los Osos Valley Road Bridge. Discharge here is expected to recharge the lower, Paso Robles Formation.

7. The most probable effluent quality standards for discharge to these locations require secondary treatment and nitrogen removal for infiltration basins and tertiary treatment for creek discharge. Discharge to Los Osos Creek will be limited to the summertime and continuity between creek discharge and Morro Bay will be prohibited.

8. Alternative sewering systems are neither cost-effective nor feasible for the Los Osos area. These systems have never been implemented for a project with a scale similar to CSA No. 9.

9. The most cost-effective, reliable wastewater collection system is by standard gravity sewers. Some small pockets of pressure sewers may be feasible as determined in final design.

10. The most cost-effective, reliable treatment system to meet the probable effluent quality requirements is sequencing batch reactors (SBRs).

11. Although the disposal/recharge site above Highland Drive appears feasible (based on field work by others), additional field testing and evaluations are required for final design to verify hydraulic loading parameters and to determine hydrogeologic impacts.

12. While some elements of the non-conventional alternatives put forth by local residents appear attractive at first glance, unfortunately they either do not provide complete solutions or are dependent upon technical conjecture. Pursuing alternative systems, such as continuing septic tanks, pumping the groundwater table down, and swapping this water with farmers for their well water up the valley, or employing new, untested technologies is not likely to result in satisfactory solutions to the problem.

13. The Recommended Plan for wastewater management for CSA No. 9 will have an estimated first cost of approximately $39 million and will require about $0.5 million per year to operate and maintain.

RECOMMENDATIONS

Based on the findings and conclusions reached during the study and presented herein, Engineering-Science recommends that the County take the following steps in the adoption and the implementation of The Recommended Plan:

1. Conduct a workshop/public hearing in conjunction with the CSA No. 9 Advisory Group to solicit comments regarding this draft report.

2. Submit The Recommended Plan to the Board of Supervisors for adoption as The Wastewater Management Plan for CSA No. 9.

3. Complete the CEQA process by directing the Morro Group to complete the EIR using The Recommended Plan as "the Project".

4. Open negotiations with landowners for purchase of both the treatment plant site and the infiltration basins sites; secure appraisals.

5. Direct ES to proceed with Phase Two and to conduct necessary additional geological and hydrogeological field work for design of the groundwater recharge system.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

PROJECT STAFF

APPENDIX B

PROJECT STAFF

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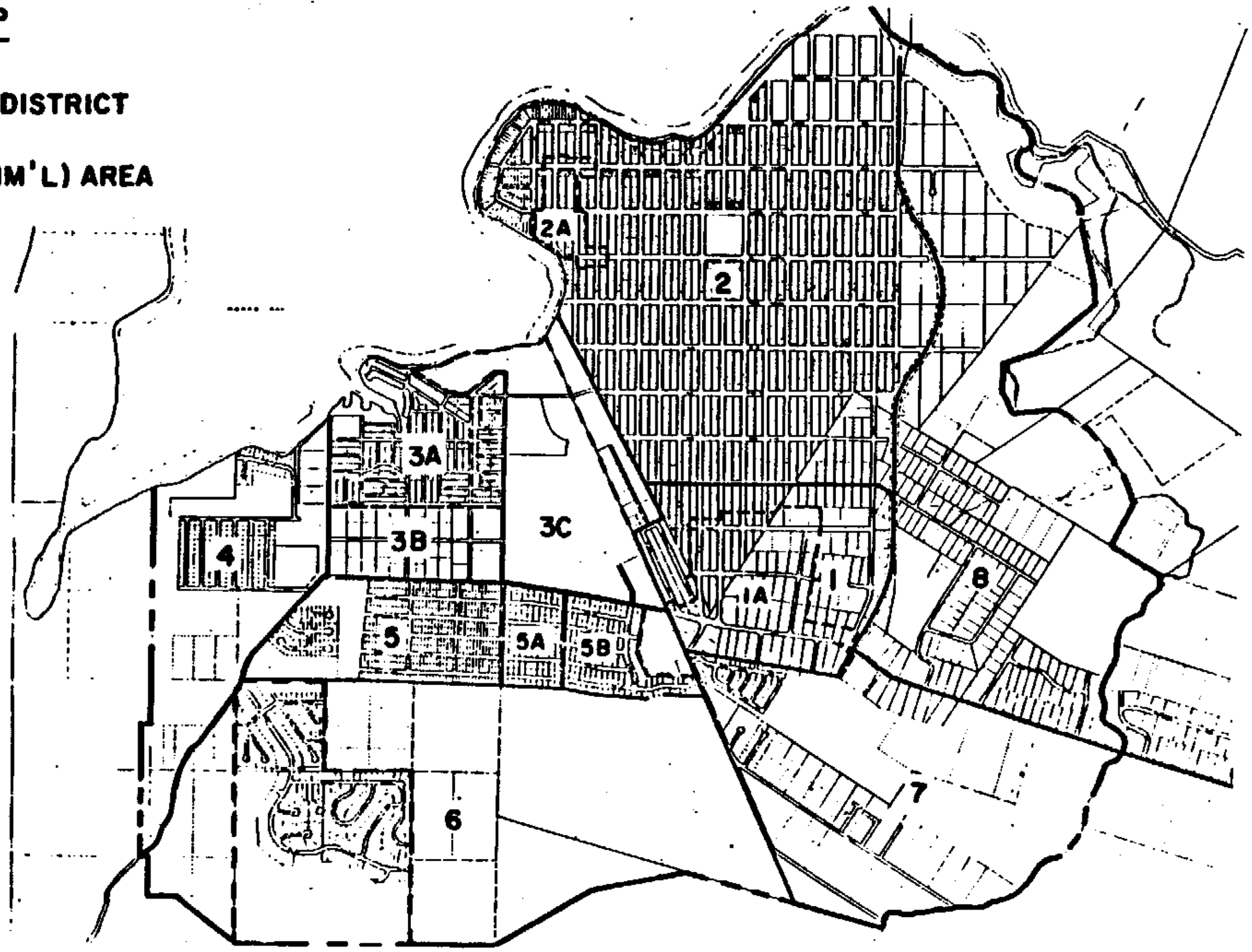
APPENDIX C

SOUTH BAY LAND USE PLAN EXCERPTS

FIGURE 2

SOUTH BAY LOCATION MAP

1. LOS OSOS VILLAGE AREA
   A - CENTRAL BUSINESS DISTRICT
2. EL MORRO AREA
   A - BAYWOOD PARK (COMM'L) AREA
3. CUESTA AREA
   A - CUESTA-BY-THE-SEA
   B - MARTIN TRACT
   C - MORRO PALISADES
4. SUNSET AREA
5. UPLAND AREA
   A - TRACT NO. 84
   B - TRACT NO. 122
6. HIGHLAND AREA
   A - CABRILLO ESTATES
7. BAYVIEW HEIGHTS AREA
8. CREEKSIDE AREA



existing urban type uses including two membership organizations serving the surrounding communities and a mobilehome and recreation vehicle park which was approved as a phased development through a conditional use permit.

Residential Rural

A portion of Clark Valley over the ridge south of Los Osos Valley has been developing with rural residential homesites on lots mostly ranging from 10 to 40 acres. Land divisions to create additional rural homesites should occur only within this defined area to ensure that surrounding agricultural uses are not converted from production. A nearby area proposed for rural residential use is at the southwest corner of Clark Valley Road and Los Osos Valley Road. An additional area east of Clark Valley Road has been designated to recognize the existing church facilities.

Residential Suburban

An area of mostly one acre lots on the north side of Los Osos Valley Road between the Los Osos Valley Memorial Cemetery and Los Osos Creek is designated residential suburban. This relatively small, isolated tract is bordered on three sides by prime agricultural land. The area is to remain outside the South Bay urban reserve line with no further expansion of suburban uses to be allowed. Moreover, substantial undeveloped suburban areas occur in the eastern part of the South Bay urban reserve area.

Another area of 1 to 5 acre homesites is at the intersection of San Luisito Creek Road and Highway 1. This area is surrounded by prime or potential prime agricultural lands and should not be allowed to expand for residential uses.

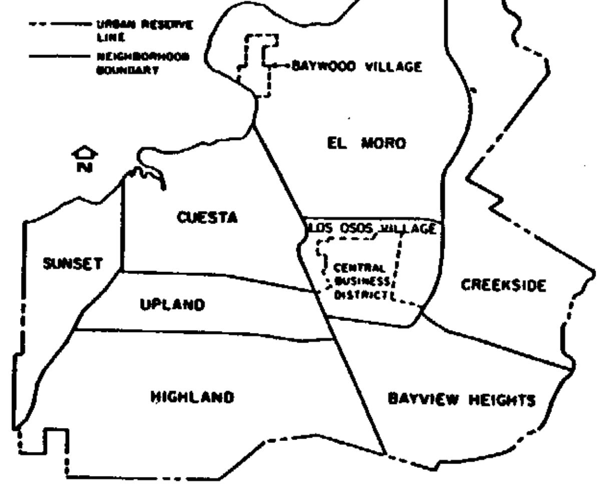
B. SOUTH BAY URBAN AREA LAND USE

The South Bay urban area is at the westerly end of Los Osos Valley and is bounded by Los Osos Creek on the east, Morro Bay and its tidelands on the north, Irish Hills on the south, and Montana de Oro and Morro Bay State Parks on the west. Although generally referred to as South Bay, the area consists of several loose-knit neighborhoods, including Baywood Park, Los Osos, and Cuesta-by-the-Sea.

The urban reserve line encompasses approximately 2,590 acres (four square miles) and allows for future growth through in-filling of existing developed areas and expansion onto adjacent vacant lands.

The South Bay area is divided into neighborhoods to show local facilities needs such as schools, parks, and convenience shopping. These neighborhood units are named simply to provide a convenient aid to location in the text discussion of land uses.

SOUTH BAY NEIGHBORHOODS



Open Space

An important area in South Bay designated Open Space is Los Osos Oak Forest, a 60 acre state park reserve with a looping footpath for observation of the pygmy oaks and other natural features. A smaller portion of the oak forest includes the dense stand of pygmy oaks preserved in open space lots within Tract 527. Another area is the northeasterly fringe area of South Bay where Los Osos Creek enters Morro Bay; this riparian and marshland area with adjacent banks is under current negotiation for addition to Morro Bay State Park.

All other sensitive resource areas (notably including the bay frontage and marshes) are now designated for passive recreation would be redesignated Open Space if acquired by government or preserved by open space easements.

Recreation

Recreation land in South Bay may be divided into active and passive recreational areas. Active recreation areas include community parks, special recreation activities and tourist lodging facilities. Passive recreation areas have limited or no access and are intended for protection of their natural biotic and scenic resources.

South Bay Park is a typical community park with picnic, sports, and playground facilities. As the community grows, another community park with picnic facilities, walking paths through both landscaped and significant native vegetation areas, and similar facilities befitting the hilly terrain and capitalizing on the scenic overlook of Morro Bay should be developed in the Highland area.

Neighborhood parks should be developed adjacent to future schools and Baywood Elementary School. This will provide recreation facilities throughout the community and allow for joint use of each facility. Additional neighborhood parks should be developed in the area west of 1st Street and south of Santa Ysabel Avenue and in the area south of the existing mobilehome parks south of Ramona Avenue.

The development of a recreational trails system, providing routes for bicycle and pedestrian-oriented activities to link parks and other scenic area is essential.

Significant tree groves within South Bay should be studied for potential public park sites before private development occurs. Native and introduced trees include oaks, willows, pines, cypress, and eucalyptus. Use of such areas may range from small parkland areas with walking and bicyle access only to fully developed neighborhood or community parks.

Among specialized recreation facilities is the 9-hole Sunset Terrace Golf Course. The 18-hole golf course in Morro Bay State Park is within a relatively short driving distance from South Bay.

A variety of outdoor activities, primary related to public access to the ocean shoreline, is provided in the nearby Montana de Oro and Morro Bay State Parks.

Other areas indicated for recreation are the bay frontage areas including the Cuesta-by the Sea, Sweet Springs, and portions of the Santa Ysabel peninsula. Each of these areas supports unique habitats that must be preserved with minimal disruption of the ecological system. These areas are included in the Sensitive Resource Area Combining Designation and are encouraged for preservation by either public acquisition or open space easements. Once this occurs, the areas would be redesignated as Open Space during the course of future general plan amendments.

Residential Rural

A small area has been designated Residential Rural in the northern portion of South Bay. This area is covered with a dense stand of pygmy oaks sloping toward the bay and the marshlands. This area is being considered for acquisition by the State Department of Parks and Recreation and would be redesignated Open Space if acquired. If acquisition does not occur, any development proposed should be through resubdivision of the area and clustering of the units in the least sensitive portion of the site.

Residential Suburban

Large suburban homesites are available in the eastern portion of the community known as the Creekside Area This area is generally bounded by South Bay Boulevard on the west, Los Osos Creek on the east and Los Osos Valley Road on the south; small areas adjoining this area lie east of South Bay Boulevard and immediately south of Los Osos Valley Road. Growth in the area has been slow. The area is characterized by a rural atmosphere and parcel sizes of one acre and larger. Significant natural features in this area include pygmy oak groves and riparian habitats along the creek. This area serves as a buffer between the more intensive urban community and adjacent agricultural areas. Incidental light agricultural uses are generally compatible with suburban residential uses; some land has the potential for productive agricultural uses, depending on the topography and water availability.

The undeveloped western portion of the Sunset and Highland areas are also designated for Residential Suburban use. Lot sizes are generally large, and access is limited to much of this area. Portions of the area support large stands of eucalyptus groves which add significantly to the character. In addition, much of this area is immediately adjacent to the habitat for the Morro Bay kangaroo rat, an endangered species.

A third area identified for Residential Suburban use is the southern hillsides of the community. The steeper portion located south of the proposed extension of South Bay Boulevard will develop at suburban densities. It is anticipated that the development will be clustered on the more gentle slopes of the area with the upper steeper slopes being preserved in their natural state.

Residential Single Family

Single family residential uses occupy more land than any other land use in South Bay and include a wide variety of residential neighborhoods ranging from the older tract areas developed in the late 1800's with undersized 25 foot wide lots, to hillside homes with lots ranging from 10,000 square feet to one acre. Each presents a unique neighborhood feeling that blends to form the community character of South Bay. Significant environmental resources are found throughout the community and serve as a scenic backdrop surrounding the periphery.

Growth is occurring through both in-filling of existing tracts on a lot-by-lot basis and lot divisions and larger subdivisions. The timing and ultimate size of the community's growth must be directly related to available water resources and other urban services. The following is a discussion of specific neighborhoods shown in Figure 1.

El Moro

The El Moro neighborhood is the largest in terms of both land area and population. Its 704 acres are bordered on the north by Morro Bay and on the south by the village area. Most of the area was subdivided in the late 1800's into lots of slightly more than 3,000 square feet with 80 foot street rights-of-way laid out in a grid pattern. Development has occurred sporadically, primarily in single family uses, and most building has occurred on multiple lots. A mixture of housing styles and sizes gives the present character to the community. Small, older homes, many of which were initially built for second homes or retirement, are scattered throughout the area. Future growth will involve in-filling of vacant lots, so the present mixture of housing types will continue; however, larger homes are being built on these small lots, often resulting in a cramped, overcrowded feeling. Of particular concern in some of this area is the lack of street trees or other features that tie the neighborhoods together. A street treatment program to break the grid pattern would improve neighborhood appearance and character.

Highland

This area lies in the southerly portion of the planning area and is only partly developed. Cabrillo Estates contains nearly all of the current population on lots exceeding 10,000 square feet. All remaining parcels in the Highland area are five acres or more in size. The Morro Palisades Development Plan involves 205 acres in the southeastern portion of the Highland area.

The Highland area is the only part of the planning area with significant slopes. Topography changes continually from the area's northerly boundary where slopes range from 0-10% to slopes in excess of 30% along the southerly boundary. The landscape of the neighborhood is dominated with vegetation. Much of the flatter area is covered with chaparral and scrub oaks, while the steepest areas are comprised of dense stands of California live oak with scattered eucalyptus and other varieties of trees. These natural vegetation areas should be retained through clustering of lots. The residential single family area is generally located north of the proposed extension of South Bay Boulevard. Minimum lot size for this area would be one half acre.

Bayview Heights

Most of the existing single family residential uses are suburban residential in character. Minimum lot sizes in this area will ultimately range from one-half acre to one acre. The present rural atmosphere should be maintained. Access to newly created parcels has been the primary concern in the area. The pattern of rights-of-way dedication has not been consistent; similarly, the lot pattern has not been consistent.

Cuesta

The Cuesta neighborhood lies just west of El Moro and north of Los Osos Valley Road and contains approximately 369 acres. Almost 30% of the area is subdivided into 3,000 to 4,000 square foot lots, another 16% is in parcels of approximately one acre, and the remainder of the area is essentially unsubdivided but planned for development under an approved development plan. Uses are almost entirely individual single family lots with the exception of Morro Shores Mobile Home Park. The approved Morro Palisades Development Plan envisions a mixture of both single and multiple family uses as well as recreational and commercial uses in the southern portion of the planning area along Los Osos Valley Road.

Parcel sizes of slightly more than one acre are predominant in the Martin Tract area situated north of Los Osos Valley Road between Pecho Road and Broderson Avenue. The plan recommends lots ranging from 10,000 to 20,000 square feet. Provision of an adequate circulation pattern in this area is essential.

Upland

This neighborhood is located just south of the Cuesta area and west of the Village area. It is the smallest neighborhood in land area with 210 acres but one of the larger in population. Lot sizes range from 5,000 to 10,000 square feet and are almost entirely developed. Views of the bay are enhanced by the terrace layout of the lot pattern.

Sunset

This neighborhood lies along the westerly perimeter of the planning area and is comprised of 219 acres. Much of the area is relatively undeveloped. Existing uses are chiefly single family residential with some multiple family adjacent to the privately owned 9-hole golf course.

Among the chief concerns in the area are tideland and marshland along the neighborhood's northerly boundary and kangaroo rat habitat on the west. In addition, significant stands of eucalyptus cover approximately 35 acres in the middle of the area. Development plans for these areas should incorporate protection of sensitive features through clustering of uses.

Residential Multi-Family

Recent high population growth rates are attributed in large part to less expensive housing costs due to single family building site availability. As this availability diminishes, the demand for multiple family units may increase. Only 25 acres are presently in multiple family residential use and is concentrated in areas identified for continuation in the same use. The plan provides for expansion of this use, but high development densities should be avoided to help protect water quality in the underlying groundwater basin. Smaller properties are restricted to low intensity multiple family residential use because of lack of space for adequate septic systems. Densities could be increased if the community is eventually sewered. Proposed multiple family residential uses are designated in the following areas:

Los Osos Village
This area lies adjacent to the central commercial or office and professional areas. In the areas with substandard lots, aggregation should be encouraged to produce adequate building sites and allow for buffering from adjacent areas. The areas east of Vista del Morro provide larger building sites where clustering of units can produce on-site open space.

Sunset
The area along Butte Drive adjacent to the golf courses has developed with single story duplexes and quadruplexes. A substantial area for multiple family dwelling is also provided at the intersection of Los Osos Valley Road and Pecho Drive. Development of this area needs to be integrated with the proposed neighborhood commercial center.

Morro Palisades
The long-range development plan between Ramona Avenue and Los Osos Valley Road indicates a mixture of single and multiple family residential uses each clustered around a common area. Multiple family areas are located: (1) north of the neighborhood commercial areas and community park and (2) south of Ramona Avenue adjacent to the mobile home development to retain portions of the site in natural features. The development plan proposes a park to serve the area, as well as a system of pedestrian and bicycle links between and within clusters of development.

El Moro
This area lies adjacent to the neighborhood commercial area along Santa Ysabel and east of Second Street. Much of this development will be in duplexes interspersed with single family residences, similar to the existing pattern. Whenever possible, substandard lots in the El Moro area should be aggregated to provide adequate building sites to permit screening and avoid rows of garages and parking areas.

Office and Professional

Office and professional uses are presently scattered throughout the community but are mostly located in the downtown commercial center. Four areas are identified for concentration of these uses. The first is immediately north and east of the central business area which will serve as a buffer between retail commercial and multiple family residential uses. The second is immediately west of the community park, adjacent to the proposed neighborhood commercial area at the intersection of Los Osos Valley Road and Ravenna Avenue extension.

A third node of Office and Professional uses is provided along El Moro Avenue as an extension of the Baywood commercial center. A fourth area identifies the existing Baywood Women's Club; however, use is limited to the existing membership organization.

Commercial Retail

Commercial land uses will become increasingly important as community growth occurs. Existing commercial uses occupy about 17 acres of land throughout the community. The plan proposes 60 acres for retail commercial uses with additional allowance for small neighborhood shopping centers.

Los Osos Village
General commercial uses are concentrated in the downtown or village area of Los Osos. Commercial development in the village has proceeded at a slow pace over the years. The village has good existing and potential circulation characteristics, served by Los Osos Valley Road, Ninth Street and South Bay Boulevard. An internal loop system may be developed around the central Village area using Santa Ynez Avenue, Los Olivos Avenue and Vista Del Morro Drive.

While larger commercial establishments such as supermarkets, financial institutions, department stores and theaters are important components, smaller-scale uses such as specialty shops, eating places and shops offering personal services certainly complement the area. Building design and landscaping should be oriented toward a plaza-like, people-oriented center.

Baywood Village

Existing uses in the Baywood Village are characteristic of the kinds of uses appropriate as neighborhood commercial. These uses presently occupy about three acres; the plan proposes about 12 acres. The natural and scenic attributes of the area make it particularly suitable not only for convenience shopping needs of nearby residents, but also for tourist-oriented needs including specialty shops, restaurants and motels or hotels. Lots are almost exclusively 25 by 125 feet in size and pose constraints to development flexibility. Complementary to the character of the area, a neighborhood park is recommended to be linked with the commercial district.

The absence of large ownerships will likely cause piecemeal and small-scale development to occur. While this may be somewhat in keeping with the neighborhood concept, it also creates circulation and parking difficulties. A program to assure orderly and attractive development of Baywood Village should include adequate parking and a unifying design theme.

Cuesta

Approximately eight acres of neighborhood commercial uses are proposed in the Cuesta neighborhood adjacent to Los Osos Valley Road. This area should be developed under an overall plan to ensure uniformity in design and compatibility with adjacent uses. Access points onto Los Osos Valley Road should be kept to a minimum.

Sunset

A small neighborhood retail commercial center is provided for at the intersection of Los Osos Valley Road and Pecho Road. The commercial area is shown as a generalized site within the property; the exact boundary will be established through the development plan review process and the LUE map will be amended at the first opportunity following approval of the Development Plan to reflect the defined boundary. The center will be integrated with the development of multiple family dwellings to the west and north.

Commercial Service

Service commercial and light industrial uses are presently scattered throughout the commercial areas. These uses should be concentrated into an area of approximately 22 acres immediately northeast of the Los Osos retail commercial area. Care must be taken to provide for more intensive traffic including supply and service trucks that frequent service commercial areas. On-site parking and loading facilities must be used to expedite traffic flow. Uses should be limited to the more essential community commercial services and uses which do not require large sites. For example, large new or used car and mobilehome sales lots, vehicle and freight terminals, and major warehousing and storage facilities would be inappropriate to the coastal community character.

Public Facilities

Only those public services and facilities that have a direct effect on land use and are publicly managed are considered. The public facilities needed for South Bay are based on policies by many public agencies. Standards for these facilities may be found in the Public Facilities section of the plan. The public facilities proposed for South Bay are noted on the Combining Designations map.

Governmental Center

Government facilities should be incorporated into a complex with such features as a community hall, library, sheriff's substation, human services center, and similar service-oriented operations. Consideration should be given to locating the complex together with the new library site adjacent to the community park.

APPENDIX D

RWQCB LETTER RE PROBABLE WASTE DISCHARGE REQUIREMENTS

STATE OF CALIFORNIA

GEORGE DEUKMEJIAN, Governor

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD —
CENTRAL COAST REGION
1102 A LAUREL LANE
SAN LUIS OBISPO, CALIFORNIA 93401
(805) 549-3147

July 17, 1985

RECEIVED
JUL 19 1985
ENGINEERING-SCIENCE, INC.
MONTEREY, CALIFORNIA

Mr. George Protopapas
County Engineer
San Luis Obispo County
County Government Center
San Luis Obispo, CA 93408

Dear Mr. Protopapas:

SUBJECT: PROBABLE WASTE DISCHARGE REQUIREMENTS FOR SLO CSA#9, LOS OSOS

Listed below, for your information, are the probable waste discharge requirements for the proposed Los Osos Treatment/Disposal Facility. These limitations assume there will be a discharge to Los Osos Creek when surface water continuity with Morro Bay does not exist, and percolation basins will be utilized when Los Osos Creek/Morro Bay continuity does exist. These limitations will be initial staff recommendations and are subject to the public review process.

Limitations are categorized as Prohibitions, Effluent Limitations, and Receiving Water Limitations as follows:

A. Prohibitions

1. The discharge of partially treated waste water is prohibited.

2. Discharge of petroleum products is prohibited.

3. Discharge of waste water to water contact recreation areas of Los Osos Creek is prohibited unless a benefit to the receiving water can be realized from the discharge.

4. Discharge of waste water to Los Osos Creek is prohibited whenever surface water continuity exists between the discharge point and Morro Bay, unless, between December 1 and April 30, total flow through the plant and precipitation on storage ponds exceeds the volume of water projected to result from design flow plus seasonal precipitation with a projected recurrence interval of 25 years.

5. The discharge of treated waste water onto land areas within 100 feet of any well used for domestic supply or irrigation of food crops is prohibited.

B. Effluent Limitations

1. Waste water discharged to Los Osos Creek shall be adequately disinfected, oxidized, coagulated, clarified, filtered wastewater. Waste water shall be considered adequately disinfected if at some location in the treatment process the median number of coliform organisms does not exceed 2.2 per-100-milliliters and the number of coliform organisms does not exceed 23 per-100-milliliters in more than one sample within any 30-day period. The median value shall be determined from the bacteriological results of the last 7 days for which analyses have been completed.

2. Waste water discharged to Los Osos Creek shall not exceed the following limitations:

Parameter	Mean	Max.
Settleable Solids, mg/l	-	0.1
BOD, mg/l	10	25
Suspended Solids, mg/l	10	15
Turbidity, JTU	2	5
Chlorine Residual, mg/l	Undetectable	
Dissolved Oxygen, mg/l	Minimum of 5.0 at all times	
pH	Within range of 6.5 to 8.3 at all times	
Oil and Grease, mg/l	10	20
Toxicity Concentration		0.59*

*No more than one of three consecutive static bioassays shall result in less than 100% survival in undiluted effluent. No single test shall ever result in less than 90% survival in undiluted effluent (.59 tu).

3. Freeboard shall exceed two feet in lagoons and ponds (unless technical justification is provided to support lesser freeboard).

4. Effluent discharged from the treatment facility shall not exceed the following limitations:

Parameter	Mean	Max.
Total Filtrable Residue, mg/l	WS + 250	WS + 500
Sodium, mg/l	" + 70	" + 140
Chloride, mg/l	" + 65	" + 130
Sulfate, mg/l	" + 40	" + 80
Total Hardness (as $CaCo_4$),mg/l	" + 30	" + 60
Total Nitrogen (Nitrate plus Nitrite plus Kjeldahl) as N, mg/l	5**	10
Settleable Solids, ml/l	0.1	0.5
BOD, mg/l	60	100
Suspended Solids, mg/l	60	100

**This limit will be based on the receiving water objective and determined after engineering justification is submitted on the treatment scheme (to give credit for treatment, operational techniques, and effect of the soil column, as appropriate.)

5. Treatment and discharge shall not cause objectionable odors.

C. Receiving Water Limitations

1. The discharge shall not cause the following limits to be exceeded in Los Osos Creek:

Constituent	Maximum, mg/l (Unless otherwise noted)
Aluminum	7.5
Arsenic	0.05
Barium	1.
Beryllium	0.15
Boron	1.25
Cadmium	0.010
Chromium	0.05
Cobalt	0.075
Copper	0.045
Fluoride	1.5
Iron	7.5
Lead	0.05
Lithium	3.75
Manganese	0.3
Mercury	0.0003
Molybdenum	0.015
Nickel	0.3
Selenium	0.01
Silver	0.05
Vanadium	0.15
Zinc	0.3
M.B.A.S.	0.2
Phenols	0.1
Polychlorinated Byphenyls	0.0003
Un-ionized Ammonia (NH_3 as N)	0.025
Phthalate Esters	0.002
Endrin	0.0002
Lindane	0.004
Methoxychlor	0.1
Toxaphene	0.005
2, 4-D	0.1
2,4,5-TP Silvex	0.01
pH	Within limit of 7.0 to 8.3 at all times, and not changed more than 0.5 units.

Temperature	Maximum increase of 5°F above natural receiving water temperature
Turbidity (NTU)	Not to exceed the following:

Natural Turbidity (NT),*NTU	Maximum Increase
< 50	20%
50 ≤NT ≤100	10 NTU
>100	10%

*"Natural Turbidity" shall be determined from receiving water samples taken upstream of the discharge point.

2. The discharge shall not cause the following limits to be exceeded in ground water:

Constituent		Maximum, mg/l (Unless otherwise noted)
Aluminum		
Arsenic		7.5
Barium		0.05
Beryllium		1.0
Boron		0.15
Cadmium		1.25
Chromium		0.010
Cobalt		0.05
Copper		0.15
Fluoride		0.075
Iron		0.3
Lead		7.5
Lithium		0.05
Manganese		3.75
Mercury		0.3
Molybdenum		0.002
Nickel		0.015
Nitrate (As N)		0.3
Selenium		5.0
Silver		0.01
Vanadium		0.05
Zinc		0.15
Endrin		3.0
Lindane		0.0002
Methoxychlor		0.004
Toxaphene		0.1
2, 4-D		0.005
2,4,5-TP Silvex		0.1
pH	Within limit of 6.5 to 8.3 at all times, and not changed more than 0.5 units.	0.01

3. The discharge to Los Osos Creek shall not cause surface waters to be greater than 15 units or 10 percent above natural background color, whichever is greater.

4. The discharge to Los Osos Creek shall not contain biostimulatory substances in concentrations which promote aquatic growths that cause nuisance or adversely affect beneficial uses.

5. The discharge shall not cause the median concentration of total coliform organisms in ground waters to be equal to or greater than 2.2 MPN/100ml over any seven day period.

6. The discharge shall not cause the nitrate-nitrogen (NO_3 as N) level of groundwater to exceed 5.0 mg/l.

7. The discharge shall not cause concentrations of chemicals and radionuclides in groundwater to exceed limits set forth in Title 22, Chapter 15, Articles 4 and 5 of the California Administrative Code.

8. The discharge to Los Osos Creek shall not cause the fecal coliform concentration, based on a minimum of not less than five samples for any 30-day period, to exceed a log mean of 200/100 ml, or cause more than ten percent of total samples during any 30-day period to exceed 400/100 ml.

9. The discharge shall not cause a violation of any applicable water quality standard for receiving waters adopted by the Regional Board or the State Water Resources Control Board as required by the Federal Clean Water Act and regulations adopted thereunder.

Very truly yours,

KENNETH R. JONES
Executive Officer

JG:sm

cc: Engineering Science, Monterey, Attn.: T.G. Cole

APPENDIX E

TITLE 22 WASTEWATER RECLAMATION CRITERIA

WASTEWATER
RECLAMATION CRITERIA

An Excerpt from the

CALIFORNIA ADMINISTRATIVE CODE
TITLE 22, DIVISION 4

ENVIRONMENTAL HEALTH

1978

STATE OF CALIFORNIA
DEPARTMENT OF HEALTH SERVICES
SANITARY ENGINEERING SECTION
2151 Berkeley Way, Berkeley 94704

INTENT OF REGULATIONS

The intent of these regulations is to establish acceptable levels of constituents of reclaimed water and to prescribe means for assurance of reliability in the production of reclaimed water in order to ensure that the use of reclaimed water for the specified purposes does not impose undue risks to health. The levels of constituents in combination with the means for assurance of reliability constitute reclamation criteria as defined in Section 13520 of the California Water Code.

As affirmed in Sections 13510 to 13512 of the California Water Code, water reclamation is in the best public interest and the policy of the State is to encourage reclamation. The reclamation criteria are intended to promote development of facilities which will assist in meeting water requirements of the State while assuring positive health protection. Appropriate surveillance and control of treatment facilities, distribution systems, and use areas must be provided in order to avoid health hazards. Precautions must be taken to avoid direct public contact with reclaimed waters which do not meet the standards specified in Article 5 for nonrestricted recreational impoundments.

TABLE OF CONTENTS

CHAPTER 3. RECLAMATION CRITERIA

Article 10. Alternative Reliability Requirements for Uses Requiring Oxidized, Disinfected Wastewater or Oxidized, Coagulated, Clarified, Filtered, Disinfected Wastewater

CHAPTER 3. RECLAMATION CRITERIA

Article 1. Definitions

60301. Definitions. (a) **Reclaimed Water.** Reclaimed water means water which, as a result of treatment of domestic wastewater, is suitable for a direct beneficial use or a controlled use that would not otherwise occur.

(b) **Reclamation Plant.** Reclamation plant means an arrangement of devices, structures, equipment, processes and controls which produce a reclaimed water suitable for the intended reuse.

(c) **Regulatory Agency.** Regulatory agency means the California Regional Water Quality Control Board in whose jurisdiction the reclamation plant is located.

(d) **Direct Beneficial Use.** Direct beneficial use means the use of reclaimed water which has been transported from the point of production to the point of use without an intervening discharge to waters of the State.

(e) **Food Crops.** Food crops mean any crops intended for human consumption.

(f) **Spray Irrigation.** Spray irrigation means application of reclaimed water to crops by spraying it from orifices in piping.

(g) **Surface Irrigation.** Surface irrigation means application of reclaimed water by means other than spraying such that contact between the edible portion of any food crop and reclaimed water is prevented.

(h) **Restricted Recreational Impoundment.** A restricted recreational impoundment is a body of reclaimed water in which recreation is limited to fishing, boating, and other non-body-contact water recreation activities.

(i) **Nonrestricted Recreational Impoundment.** A nonrestricted recreational impoundment is an impoundment of reclaimed water in which no limitations are imposed on body-contact water sport activities.

(j) **Landscape Impoundment.** A landscape impoundment is a body of reclaimed water which is used for aesthetic enjoyment or which otherwise serves a function not intended to include public contact.

(k) **Approved Laboratory Methods.** Approved laboratory methods are those specified in the latest edition of "Standard Methods for the Examination of Water and Wastewater", prepared and published jointly by the American Public Health Association, the American Water Works Association, and the Water Pollution Control Federation and which are conducted in laboratories approved by the State Department of Health.

(l) **Unit Process.** Unit process means an individual stage in the wastewater treatment sequence which performs a major single treatment operation.

(m) **Primary Effluent.** Primary effluent is the effluent from a wastewater treatment process which provides removal of sewage solids so that it contains not more than 0.5 milliliter per liter per hour of settleable solids as determined by an approved laboratory method.

(n) **Oxidized Wastewater.** Oxidized wastewater means wastewater in which the organic matter has been stabilized, is nonputrescible, and contains dissolved oxygen.

(o) **Biological Treatment.** Biological treatment means methods of wastewater treatment in which bacterial or biochemical action is intensified as a means of producing an oxidized wastewater.

(p) **Secondary Sedimentation.** Secondary sedimentation means the removal by gravity of settleable solids remaining in the effluent after the biological treatment process.

(q) **Coagulated Wastewater.** Coagulated wastewater means oxidized wastewater in which colloidal and finely divided suspended matter have been destabilized and agglomerated by the addition of suitable floc-forming chemicals or by an equally effective method.

(r) **Filtered Wastewater.** Filtered wastewater means an oxidized, coagulated, clarified wastewater which has been passed through natural undisturbed soils or filter media, such as sand or diatomaceous earth, so that the turbidity as determined by an approved laboratory method does not exceed an average operating turbidity of 2 turbidity units and does not exceed 5 turbidity units more than 5 percent of the time during any 24-hour period.

(s) **Disinfected Wastewater.** Disinfected wastewater means wastewater in which the pathogenic organisms have been destroyed by chemical, physical or biological means.

(t) **Multiple Units.** Multiple units means two or more units of a treatment process which operate in parallel and serve the same function.

(u) **Standby Unit Process.** A standby unit process is an alternate unit process or an equivalent alternative process which is maintained in operable condition and which is capable of providing comparable treatment for the entire design flow of the unit for which it is a substitute.

(v) **Power Source.** Power source means a source of supplying energy to operate unit processes.

(w) **Standby Power Source.** Standby power source means an automatically actuated self-starting alternate energy source maintained in immediately operable condition and of sufficient capacity to provide necessary service during failure of the normal power supply.

(x) **Standby Replacement Equipment.** Standby replacement equipment means reserve parts and equipment to replace broken-down or worn-out units which can be placed in operation within a 24-hour period.

(y) **Standby Chlorinator.** A standby chlorinator means a duplicate chlorinator for reclamation plants having one chlorinator and a duplicate of the largest unit for plants having multiple chlorinator units.

(z) **Multiple Point Chlorination.** Multiple point chlorination means that chlorine will be applied simultaneously at the reclamation plant and at subsequent chlorination stations located at the use area and/or some intermediate point. It does not include chlorine application for odor control purposes.

(aa) **Alarm.** Alarm means an instrument or device which continuously monitors a specific function of a treatment process and automatically gives warning of an unsafe or undesirable condition by means of visual and audible signals.

(bb) **Person.** Person also includes any private entity, city, county, district, the State or any department or agency thereof.

NOTE: Authority cited: Section 208, Health and Safety Code and Section 13521, Water Code. Reference: Section 13521, Water Code.

History: 1. New Chapter 4 (§§ 60301–60357, not consecutive) filed 4-2-75; effective thirtieth day thereafter (Register 75, No. 14).
2. Renumbering of Chapter 4 (Sections 60301–60357, not consecutive) to Chapter 3 (Sections 60301–60357, not consecutive), filed 10-14-77; effective thirtieth day thereafter (Register 77, No. 42).

Article 2. Irrigation of Food Crops

60303. **Spray Irrigation.** Reclaimed water used for the spray irrigation of food crops shall be at all times an adequately disinfected, oxidized, coagulated, clarified, filtered wastewater. The wastewater shall be considered adequately disinfected if at some location in the treatment process the median number of coliform organisms does not exceed 2.2 per 100 milliliters and the number of coliform organisms does not exceed 23 per 100 milliliters in more than one sample within any 30-day period. The median value shall be determined from the bacteriological results of the last 7 days for which analyses have been completed.

60305. **Surface Irrigation.** (a) Reclaimed water used for surface irrigation of food crops shall be at all times an adequately disinfected, oxidized wastewater. The wastewater shall be considered adequately disinfected if at some location in the treatment process the median number of coliform organisms does not exceed 2.2 per 100 milliliters, as determined from the bacteriological results of the last 7 days for which analyses have been completed.

(b) Orchards and vineyards may be surface irrigated with reclaimed water that has the quality at least equivalent to that of primary effluent provided that no fruit is harvested that has come in contact with the irrigating water or the ground.

60307. **Exceptions.** Exceptions to the quality requirements for reclaimed water used for irrigation of food crops may be considered by the State Department of Health on an individual case basis where the reclaimed water is to be used to irrigate a food crop which must undergo extensive commercial, physical or chemical processing sufficient to destroy pathogenic agents before it is suitable for human consumption.

Article 3. Irrigation of Fodder, Fiber, and Seed Crops

60309. Fodder, Fiber, and Seed Crops. Reclaimed water used for the surface or spray irrigation of fodder, fiber, and seed crops shall have a level of quality no less than that of primary effluent.

60311. Pasture for Milking Animals. Reclaimed water used for the irrigation of pasture to which milking cows or goats have access shall be at all times an adequately disinfected, oxidized wastewater. The wastewater shall be considered adequately disinfected if at some location in the treatment process the median number of coliform organisms does not exceed 23 per 100 milliliters, as determined from the bacteriological results of the last 7 days for which analyses have been completed.

Article 4. Landscape Irrigation

60313. Landscape Irrigation. (a) Reclaimed water used for the irrigation of golf courses, cemeteries, freeway landscapes, and landscapes in other areas where the public has similar access or exposure shall be at all times an adequately disinfected, oxidized wastewater. The wastewater shall be considered adequately disinfected if the median number of coliform organisms in the effluent does not exceed 23 per 100 milliliters, as determined from the bacteriological results of the last 7 days for which analyses have been completed, and the number of coliform organisms does not exceed 240 per 100 milliliters in any two consecutive samples.

(b) Reclaimed water used for the irrigation of parks, playgrounds, schoolyards, and other areas where the public has similar access or exposure shall be at all times an adequately disinfected, oxidized, coagulated, clarified, filtered wastewater or a wastewater treated by a sequence of unit processes that will assure an equivalent degree of treatment and reliability. The wastewater shall be considered adequately disinfected if the median number of coliform organisms in the effluent does not exceed 2.2 per 100 milliliters, as determined from the bacteriological results of the last 7 days for which analyses have been completed, and the number of coliform organisms does not exceed 23 per 100 milliliters in any sample.

NOTE: Authority cited: Section 208, Health and Safety Code and Section 13521, Water Code. Reference: Section 13520, Water Code.

History: 1. Amendment filed 9-22-78; effective thirtieth day thereafter (Register 78, No. 38).

Article 5. Recreational Impoundments

60315. Nonrestricted Recreational Impoundment. Reclaimed water used as a source of supply in a nonrestricted recreational impoundment shall be at all times an adequately disinfected, oxidized, coagulated, clarified, filtered wastewater. The wastewater shall be considered adequately disinfected if at some location in the treatment process the median number of coliform organisms does not exceed 2.2 per 100 milliliters and the number of coliform organisms does not exceed 23 per 100 milliliters in more than one sample within any 30-day period. The median value shall be determined from the bacteriological results of the last 7 days for which analyses have been completed.

60317. Restricted Recreational Impoundment. Reclaimed water used as a source of supply in a restricted recreational impoundment shall be at all times an adequately disinfected, oxidized wastewater. The wastewater shall be considered adequately disinfected if at some location in the treatment process the median number of coliform organisms does not exceed 2.2 per 100 milliliters, as determined from the bacteriological results of the last 7 days for which analyses have been completed.

60319. Landscape Impoundment. Reclaimed water used as a source of supply in a landscape impoundment shall be at all times an adequately disinfected, oxidized wastewater. The wastewater shall be considered adequately disinfected if at some location in the treatment process the median number of coliform organisms does not exceed 23 per 100 milliliters, as determined from the bacteriological results of the last 7 days for which analyses have been completed.

Article 5.1. Groundwater Recharge

60320. Groundwater Recharge. (a) Reclaimed water used for groundwater recharge of domestic water supply aquifers by surface spreading shall be at all times of a quality that fully protects public health. The State Department of Health Services' recommendations to the Regional Water Quality Control Boards for proposed groundwater recharge projects and for expansion of existing projects will be made on an individual case basis where the use of reclaimed water involves a potential risk to public health.

(b) The State Department of Health Services' recommendations will be based on all relevant aspects of each project, including the following factors: treatment provided; effluent quality and quantity; spreading area operations; soil characteristics; hydrogeology; residence time; and distance to withdrawal.

(c) The State Department of Health Services will hold a public hearing prior to making the final determination regarding the public health aspects of each groundwater recharge project. Final recommendations will be submitted to the Regional Water Quality Control Board in an expeditious manner.

NOTE: Authority cited: Section 208, Health and Safety Code and Section 13521, Water Code. Reference: Section 13520, Water Code.

History: 1. New Article 5.1 (Section 60320) filed 9-22-78; effective thirtieth day thereafter (Register 78, No. 38).

Article 5.5. Other Methods of Treatment

60320.5. Other Methods of Treatment. Methods of treatment other than those included in this chapter and their reliability features may be accepted if the applicant demonstrates to the satisfaction of the State Department of Health that the methods of treatment and reliability features will assure an equal degree of treatment and reliability.

NOTE: Authority cited: Section 208, Health and Safety Code and Section 13521, Water Code. Reference: Section 13520, Water Code.

History: 1. Renumbering of Article 11 (Section 60357) to Article 5.5 (Section 60320.5) filed 9-22-78; effective thirtieth day thereafter (Register 78, No. 38).

(Next page is 1605)

Article 6. Sampling and Analysis

60321. Sampling and Analysis. (a) Samples for settleable solids and coliform bacteria, where required, shall be collected at least daily and at a time when wastewater characteristics are most demanding on the treatment facilities and disinfection procedures. Turbidity analysis, where required, shall be performed by a continuous recording turbidimeter.

(b) For uses requiring a level of quality no greater than that of primary effluent, samples shall be analyzed by an approved laboratory method of settleable solids.

(c) For uses requiring an adequately disinfected, oxidized wastewater, samples shall be analyzed by an approved laboratory method for coliform bacteria content.

(d) For uses requiring an adequately disinfected, oxidized, coagulated, clarified, filtered wastewater, samples shall be analyzed by approved laboratory methods for turbidity and coliform bacteria content.

Article 7. Engineering Report and Operational Requirements

60323. Engineering Report. (a) No person shall produce or supply reclaimed water for direct reuse from a proposed water reclamation plant unless he files an engineering report.

(b) The report shall be prepared by a properly qualified engineer registered in California and experienced in the field of wastewater treatment, and shall contain a description of the design of the proposed reclamation system. The report shall clearly indicate the means for compliance with these regulations and any other features specified by the regulatory agency.

(c) The report shall contain a contingency plan which will assure that no untreated or inadequately-treated wastewater will be delivered to the use area.

60325. Personnel. (a) Each reclamation plant shall be provided with a sufficient number of qualified personnel to operate the facility effectively so as to achieve the required level of treatment at all times.

(b) Qualified personnel shall be those meeting requirements established pursuant to Chapter 9 (commencing with Section 13625) of the Water Code.

60327. Maintenance. A preventive maintenance program shall be provided at each reclamation plant to ensure that all equipment is kept in a reliable operating condition.

60329. Operating Records and Reports. (a) Operating records shall be maintained at the reclamation plant or a central depository within the operating agency. These shall include: all analyses specified in the reclamation criteria; records of operational problems, plant and equipment breakdowns, and diversions to emergency storage or disposal; all corrective or preventive action taken.

(b) Process or equipment failures triggering an alarm shall be recorded and maintained as a separate record file. The recorded information shall include the time and cause of failure and corrective action taken.

(c) A monthly summary of operating records as specified under (a) of this section shall be filed monthly with the regulatory agency.

(d) Any discharge of untreated or partially treated wastewater to the use area, and the cessation of same, shall be reported immediately by telephone to the regulatory agency, the State Department of Health, and the local health officer.

60331. Bypass. There shall be no bypassing of untreated or partially treated wastewater from the reclamation plant or any intermediate unit processes to the point of use.

Article 8. General Requirements of Design

60333. Flexibility of Design. The design of process piping, equipment arrangement, and unit structures in the reclamation plant must allow for efficiency and convenience in operation and maintenance and provide flexibility of operation to permit the highest possible degree of treatment to be obtained under varying circumstances.

60335. Alarms. (a) Alarm devices required for various unit processes as specified in other sections of these regulations shall be installed to provide warning of:

(1) Loss of power from the normal power supply.
(2) Failure of a biological treatment process.
(3) Failure of a disinfection process.
(4) Failure of a coagulation process.
(5) Failure of a filtration process.
(6) Any other specific process failure for which warning is required by the regulatory agency.

(b) All required alarm devices shall be independent of the normal power supply of the reclamation plant.

(c) The person to be warned shall be the plant operator, superintendent, or any other responsible person designated by the management of the reclamation plant and capable of taking prompt corrective action.

(d) Individual alarm devices may be connected to a master alarm to sound at a location where it can be conveniently observed by the attendant. In case the reclamation plant is not attended full time, the alarm(s) shall be connected to sound at a police station, fire station or other full-time service unit with which arrangements have been made to alert the person in charge at times that the reclamation plant is unattended.

60337. Power Supply. The power supply shall be provided with one of the following reliability features:

(a) Alarm and standby power source.

(b) Alarm and automatically actuated short-term retention or disposal provisions as specified in Section 60341.

(c) Automatically actuated long-term storage or disposal provisions as specified in Section 60341.

Article 9. Alternative Reliability Requirements for Uses Permitting Primary Effluent

60339. Primary Treatment. Reclamation plants producing reclaimed water exclusively for uses for which primary effluent is permitted shall be provided with one of the following reliability features:

(a) Multiple primary treatment units capable of producing primary effluent with one unit not in operation.

(b) Long-term storage or disposal provisions as specified in Section 60341.

Article 10. Alternative Reliability Requirements for Uses Requiring Oxidized, Disinfected Wastewater or Oxidized, Coagulated, Clarified, Filtered, Disinfected Wastewater

60341. Emergency Storage or Disposal. (a) Where short-term retention or disposal provisions are used as a reliability feature, these shall consist of facilities reserved for the purpose of storing or disposing of untreated or partially treated wastewater for at least a 24-hour period. The facilities shall include all the necessary diversion devices, provisions for odor control, conduits, and pumping and pump back equipment. All of the equipment other than the pump back equipment shall be either independent of the normal power supply or provided with a standby power source.

(b) Where long-term storage or disposal provisions are used as a reliability feature, these shall consist of ponds, reservoirs, percolation areas, downstream sewers leading to other treatment or disposal facilities or any other facilities reserved for the purpose of emergency storage or disposal of untreated or partially treated wastewater. These facilities shall be of sufficient capacity to provide disposal or storage of wastewater for at least 20 days, and shall include all the necessary diversion works, provisions for odor and nuisance control, conduits, and pumping and pump back equipment. All of the equipment other than the pump back equipment shall be either independent of the normal power supply or provided with a standby power source.

(c) Diversion to a less demanding reuse is an acceptable alternative to emergency disposal of partially treated wastewater provided that the quality of the partially treated wastewater is suitable for the less demanding reuse.

(d) Subject to prior approval by the regulatory agency, diversion to a discharge point which requires lesser quality of wastewater is an acceptable alternative to emergency disposal of partially treated wastewater.

(e) Automatically actuated short-term retention or disposal provisions and automatically actuated long-term storage or disposal provisions shall include, in addition to provisions of (a), (b), (c), or (d) of this section, all the necessary sensors, instruments, valves and other devices to enable fully automatic diversion of untreated or partially treated wastewater to approved emergency storage or disposal in the event of failure of a treatment process, and a manual reset to prevent automatic restart until the failure is corrected.

60343. Primary Treatment. All primary treatment unit processes shall be provided with one of the following reliability features:

(a) Multiple primary treatment units capable of producing primary effluent with one unit not in operation.
(b) Standby primary treatment unit process.
(c) Long-term storage or disposal provisions.

60345. Biological Treatment. All biological treatment unit processes shall be provided with one of the following reliability features:

(a) Alarm and multiple biological treatment units capable of producing oxidized wastewater with one unit not in operation.
(b) Alarm, short-term retention or disposal provisions, and standby replacement equipment.
(c) Alarm and long-term storage or disposal provisions.
(d) Automatically actuated long-term storage or disposal provisions.

60347. Secondary Sedimentation. All secondary sedimentation unit processes shall be provided with one of the following reliability features:

(a) Multiple sedimentation units capable of treating the entire flow with one unit not in operation.
(b) Standby sedimentation unit process.
(c) Long-term storage or disposal provisions.

60349. Coagulation.

(a) All coagulation unit processes shall be provided with the following mandatory features for uninterrupted coagulant feed:

(1) Standby feeders,
(2) Adequate chemical stowage and conveyance facilities,
(3) Adequate reserve chemical supply, and
(4) Automatic dosage control.

(b) All coagulation unit processes shall be provided with one of the following reliability features:

(1) Alarm and multiple coagulation units capable of treating the entire flow with one unit not in operation;
(2) Alarm, short-term retention or disposal provisions, and standby replacement equipment;
(3) Alarm and long-term storage or disposal provisions;
(4) Automatically actuated long-term storage or disposal provisions, or
(5) Alarm and standby coagulation process.

60351. Filtration. All filtration unit processes shall be provided with one of the following reliability features:

(a) Alarm and multiple filter units capable of treating the entire flow with one unit not in operation.
(b) Alarm, short-term retention or disposal provisions and standby replacement equipment.

(c) Alarm and long-term storage or disposal provisions.
(d) Automatically actuated long-term storage or disposal provisions.
(e) Alarm and standby filtration unit process.

60353. Disinfection.

(a) All disinfection unit processes where chlorine is used as the disinfectant shall be provided with the following features for uninterrupted chlorine feed:

(1) Standby chlorine supply,
(2) Manifold systems to connect chlorine cylinders,
(3) Chlorine scales, and
(4) Automatic devices for switching to full chlorine cylinders.

Automatic residual control of chlorine dosage, automatic measuring and recording of chlorine residual, and hydraulic performance studies may also be required.

(b) All disinfection unit processes where chlorine is used as the disinfectant shall be provided with one of the following reliability features:

(1) Alarm and standby chlorinator;
(2) Alarm, short-term retention or disposal provisions, and standby replacement equipment;
(3) Alarm and long-term storage or disposal provisions;
(4) Automatically actuated long-term storage or disposal provisions; or
(5) Alarm and multiple point chlorination, each with independent power source, separate chlorinator, and separate chlorine supply.

60355. Other Alternatives to Reliability Requirements. Other alternatives to reliability requirements set forth in Articles 8 to 10 may be accepted if the applicant demonstrates to the satisfaction of the State Department of Health that the proposed alternative will assure an equal degree of reliability.

(Next page is 1701)

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APPENDIX F

PRESSURE SEWER INSTALLATIONS

```
****
 REC01 = 267;
 SYSTEM.NAME = Outlet Bay WSD;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = Rt. 5, Box 138;
    ADDRESS = Sandpoint;
    STATE = ID;
    ZIP = 83864;
 CONSULTANT;
    CONSULTING.ENGR = JUB Engineers;
    ADDRESS = 2005 Ironwood Parkway;
    ADDRESS = Coeur d'Alene;
    STATE = ID;
    ZIP = 83814;
 TECHNOLOGY;
    COLLECTION = STEPS;
    TREATMENT = Lagoons, Spray irrigation;
 DATE.OPERATIONAL = 1984;
;
****
 REC01 = 266;
 SYSTEM.NAME = Pinto Point WSD;
 OPERATOR;
    CONTACT = Roy Olson;
    ADDRESS = 4227 Wall Street;
    ADDRESS = Spokane;
    STATE = WA;
    ZIP = 99205;
 CONSULTANT;
    CONSULTING.ENGR = LePard & Frame, Inc.;
    ADDRESS = 603 N 4th Street;
    ADDRESS = Coeur d'Alene;
    STATE = ID;
    ZIP = 83814;
 TECHNOLOGY;
    COLLECTION = STEPS;
    TREATMENT = Community Drainfield;
 DATE.OPERATIONAL = 1984;
 COMMENTS = Serves 53 homes;
;
****
 REC01 = 265;
 SYSTEM.NAME = Southside Sewer District;
 CONSULTANT;
    CONSULTING.ENGR = Tucker Engineering;
    ADDRESS = 617 Church Street;
    ADDRESS = Sandpoint;
    STATE = ID;
    ZIP = 83864;
 TECHNOLOGY;
    COLLECTION = STEPS;
    TREATMENT = Lagoons, Spray Irrigation;
 DATE.OPERATIONAL = 1977;
 COMMENTS = Serves about 130 homes;
```

```
   STATE = ID;
   ZIP = 83814;
 TECHNOLOGY;
   COLLECTION = STEP;
   TREATMENT = SOIL ABSORPTION ;
 DATE.OPERATIONAL = 1981;
 COMMENTS = 0.023 MGD;
;
****
 REC01 = 9;
 SYSTEM.NAME = VENTURA COUNTY ;
 OPERATOR;
   CONTACT = JERRY NOWAK, DEPUTY DIRECTOR OF PUBLIC WORKS;
   ADDRESS = FLOOD CONTROL AND WATER RESOURCES DEPT.;
   ADDRESS = PUBLIC WORKS AGENCY;
   ADDRESS = COUNTY OF VENTURA;
   ADDRESS = 800 SOUTH VICTORIA AVE.;
   ADDRESS = VENTURA;
   STATE = CA;
   ZIP = 93009;
 TECHNOLOGY;
   COLLECTION = STEP ;
   TREATMENT = AERATED LAGOON;
 DATE.OPERATIONAL = 1983;
 COMMENTS = 2,241 PEOPLE, 0.2 MGD;
;
```

```
 TECHNOLOGY;
    COLLECTION = STEP ;
 DATE.OPERATIONAL = 1982;
 COMMENTS = 0.08 MGD;
;
****
 RECO1 = 34;
 SYSTEM.NAME = BLACK BUTTE ;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = BLACK BUTTE;
    STATE = OR;
    ZIP = 97701;
 CONSULTANT;
    CONSULTING.ENGR = CENTURY WEST ENGINEERS;
    ADDRESS = BEND;
    STATE = OR;
    ZIP = 97702;
 TECHNOLOGY;
    COLLECTION = STEP;
 DATE.OPERATIONAL = 1976;
;
****
 RECO1 = 33;
 SYSTEM.NAME = BEND ;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = BEND;
    STATE = OR;
    ZIP = 97701;
 CONSULTANT;
    CONSULTING.ENGR = C & G ENGINEERS;
    ADDRESS = SALEM;
    STATE = OR;
    ZIP = 97302;
 TECHNOLOGY;
    COLLECTION = STEP , VACUUM SEWER;
 DATE.OPERATIONAL = 1980;
 COMMENTS = PRESSURE SEWER SYSTEM COLLECTS DOMESTIC SEPTIC TANK
            EFFLUENT AND VACUUM SYSTEM COLLECTS RAW DOMESTIC
            SEWAGE.  EACH SYSTEM COLLECTS SEWAGE FROM 11 HOMES
            AND DISCHARGES INTO EXITING GRAVITY SEWER MAINS;
;
****
 RECO1 = 32;
 SYSTEM.NAME = Avery WSD;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = Box 133;
    ADDRESS = Avery;
    STATE = ID;
    ZIP = 83802;
 CONSULTANT;
    CONSULTING.ENGR = V. DAVID WELCH ENGINEERS;
    ADDRESS = COEUR D'ALENE;
```

```
   ADDRESS = COEUR D'ALENE;
   STATE = ID;
   ZIP = 83814;
 TECHNOLOGY;
   COLLECTION = STEP ;
   TREATMENT = LAGOON;
   TREATMENT = Land Application (Spray irrigation);
 DATE.OPERATIONAL = 1978;
 COMMENTS = 0.023 MGD;
;
****
 REC01 = 54;
 SYSTEM.NAME = KALLISPELL BAY SEWAGE WORKS ;
 OPERATOR;
   CONTACT = WASTEWATER SYSTEM OPERATOR;
   ADDRESS = PRIEST RIVER;
   STATE = ID;
 CONSULTANT;
   CONSULTING.ENGR = K.A. DURTSCHI & ASSOC. ;
   ADDRESS = COEUR D'ALENE;
   STATE = ID;
   ZIP = 83814;
 TECHNOLOGY;
   COLLECTION = STEP ;
   TREATMENT = LAGOON;
 DATE.OPERATIONAL = 1974;
 COMMENTS = 0.056 MGD;
;
****
 REC01 = 53;
 SYSTEM.NAME = Coolin WSD;
 OPERATOR;
   CONTACT = WASTEWATER SYSTEM OPERATOR;
   ADDRESS = Coolin;
   STATE = ID;
 CONSULTANT;
   CONSULTING.ENGR = K.A. DURTSCHI & ASSOC. ;
   ADDRESS = COEUR D'ALENE;
   STATE = ID;
   ZIP = 83814;
 TECHNOLOGY;
   COLLECTION = STEP ;
   TREATMENT = LAGOON ;
   TREATMENT = Land application (spray irrigation);
 DATE.OPERATIONAL = 1972;
 COMMENTS = 0.082 MGD;
;
****
 REC01 = 35;
 SYSTEM.NAME = EAST SOUND ;
 OPERATOR;
   CONTACT = WASTEWATER SYSTEM OPERATOR;
   ADDRESS = EAST SOUND;
   STATE = WA;
   ZIP = 98245;
```

```
    ADDRESS = Bella Vista;
    STATE = AR;
    ZIP = 72712;
 TECHNOLOGY;
    COLLECTION = STEP ;
    TREATMENT = EXTENDED AERATION;
 DATE.OPERATIONAL = 1978;
 COMMENTS = 0.400 MGD;
;
****
 REC01 = 103;
 SYSTEM.NAME = RINGWOOD BOROUGH SEWAGE AUTHORITY;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = RINGWOOD;
    STATE = NJ;
    ZIP = 07456;
 TECHNOLOGY;
    COLLECTION = STEP ;
    TREATMENT = SMALL-DIAMETER GRAVITY SEWER;
 DATE.OPERATIONAL = 1982;
 COMMENTS = < 0.175 MGD;
;
****
 REC01 = 56;
 SYSTEM.NAME = City of Harrison;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = Box 73;
    ADDRESS = Harrison;
    STATE = ID;
    ZIP = 83833;
 CONSULTANT;
    CONSULTING.ENGR = URS ENGINEERS;
    ADDRESS = SEATTLE;
    STATE = WA;
    ZIP = 98101;
 TECHNOLOGY;
    COLLECTION = STEP ;
    TREATMENT = AERATED LAGOON;
    TREATMENT = Stream discharge;
 DATE.OPERATIONAL = 1977;
 COMMENTS = 0.024 MGD;
;
****
 REC01 = 55;
 SYSTEM.NAME = Bottle Bay WSD;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = Box 668;
    ADDRESS = Sandpoint;
    STATE = ID;
    ZIP = 83860;
 CONSULTANT;
    CONSULTING.ENGR = K.A.  DURTSCHI & ASSOC.;
```

```
    STATE = NC;
    ZIP = 28201;
  TECHNOLOGY;
    COLLECTION = STEP ;
    TREATMENT = LAGOON;
  DATE.OPERATIONAL = 1980;
  COMMENTS = 0.25 MGD;
;
****
  REC01 = 127;
  SYSTEM.NAME = GRANDVIEW LAKE (PRIVATE UTILITY);
  OPERATOR;
    CONTACT = MR. JAMES MOORE;
    ADDRESS = COLUMBUS;
    STATE = IN;
    ZIP = 47201;
  CONSULTANT;
    CONSULTING.ENGR = FREESE AND ABPLANALP;
    ADDRESS = FRANKLIN;
    STATE = IN;
    ZIP = 46131;
  TECHNOLOGY;
    COLLECTION = GRINDER PUMP PRESSURE SEWER, STEP;
    TREATMENT = AERATED LAGOON;
  DATE.OPERATIONAL = 1970;
  COMMENTS = 200 HOMES, 5 MILES WEST OF COLUMBUS, IN, PHONE (812)
   342-6431;
;
****
  REC01 = 119;
  SYSTEM.NAME = GARDINER ;
  OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = GARDINER;
    STATE = NY;
    ZIP = 12525;
  CONSULTANT;
    CONSULTING.ENGR = ERICKSON AND SILBER;
  TECHNOLOGY;
    COLLECTION = STEP ;
    TREATMENT = INTERMITTENT SAND FILTER;
  DATE.OPERATIONAL = 1982;
  COMMENTS = 20 HOMES, 0.05 MGD;
;
****
  REC01 = 109;
  SYSTEM.NAME = .;
  OPERATOR;
    CONTACT = David Thrasher;
    ADDRESS = ECCO Services;
    ADDRESS = Bella Vista;
    STATE = AR;
    ZIP = 72712;
  CONSULTANT;
    CONSULTING.ENGR = COOPER CONSULTANTS;
```

```
 TECHNOLOGY;
    COLLECTION = STEP pressure sewer;
    TREATMENT = Lagoon;
 COMMENTS = 31 homes;
;
****
 REC01 = 174;
 SYSTEM.NAME = PORT CHARLOTTE ;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = PORT CHARLOTTE;
    STATE = FL;
    ZIP = 33950;
 CONSULTANT;
    CONSULTING.ENGR = GENERAL DEVELOPMENT UTILITIES ENGINEERS;
    ADDRESS = 1111 SOUTH BAYSHORE DR. ;
    ADDRESS = MIAMI;
    STATE = FL;
    ZIP = 33131;
 TECHNOLOGY;
    COLLECTION = STEP ;
    TREATMENT = EXTENDED AERATIONS;
 DATE.OPERATIONAL = 1972;
 COMMENTS = 62 UNITS, PACKAGE PLANT TREATS SEPTIC TANK EFFLUENT ONLY;
;
****
 REC01 = 173;
 SYSTEM.NAME = PORT ST. LUCIE ;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = PORT ST. LUCIE;
    STATE = FL;
    ZIP = 33450;
 CONSULTANT;
    CONSULTING.ENGR = GENERAL DEVELOPMENT UTILITIES ENGINEERS;
    ADDRESS = 1111 SOUTH BAYSHORE DR.        ADDRESS = MIAMI;
    STATE = FL;
    ZIP = 33131;
 TECHNOLOGY;
    COLLECTION = STEP ;
    TREATMENT = EXTENDED AERATION, PACKAGE TREATMENT PLANT;
 DATE.OPERATIONAL = 1972;
 COMMENTS = 191 UNITS, DISCHARGES TO CONVENTIONAL GRAVITY SEWER;
;
****
 REC01 = 149;
 SYSTEM.NAME = UNION COUNTY ;
 OPERATOR;
    CONTACT = WASTEWATER SYSTEM OPERATOR;
    ADDRESS = WAXHAW;
    STATE = NC;
    ZIP = 28173;
 CONSULTANT;
    CONSULTING.ENGR = H.D. & R. ;
    ADDRESS = CHARLOTTE;
```

```
 COMMENTS = One pump per 2-4 dwellings;
;
****
 REC01 = 227;
 SYSTEM.NAME = Westboro;
 OPERATOR;
   CONTACT = WASTEWATER SYSTEM OPERATOR;
   ADDRESS = Westboro;
   STATE = WI;
   ZIP = 54490;
 CONSULTANT;
   CONSULTING.ENGR = Richard J. Otis;
   ADDRESS = P.O. Box 9443;
   ADDRESS = Madison;
   STATE = WI;
   ZIP = 53715;
 TECHNOLOGY;
   COLLECTION = Small diameter gravity sewer, step units;
   TREATMENT = Septic tanks, soil absorption fields;
 DATE.OPERATIONAL = .;
 COMMENTS = 30,000 gal per day, 69 connections.  Each home has 1000 gal
septic tank.  4 PVC gravity sewers connect homes with 3 community lift
stations.  30 connections reqUIRE STEP UNITS.  THREE 100 X 150
ABSORPTION FIELDS GIVE TREATMENT AT A LOADING OF 0.7 GPD/FT;
;
****
 REC01 = 223;
 SYSTEM.NAME = Fountain Run;
 OPERATOR;
   CONTACT = Fountain Run;
   STATE = KY;
   ZIP = 42133;
 CONSULTANT;
   CONSULTING.ENGR = Parrott, Ely & Hurt;
   ADDRESS = Consulting Engineers, Inc.;
   ADDRESS = Lexington;
   STATE = KY;
   ZIP = 40302;
 TECHNOLOGY;
   COLLECTION = STEP pressure sewer;
   TREATMENT = Community absorption trenches;
 DATE.OPERATIONAL = n/a;
 COMMENTS = 122 customers;
;
****
 REC01 = 222;
 SYSTEM.NAME = Grady W. Taylor Subdivision;
 OPERATOR;
   CONTACT = Mt. Andrew;
   STATE = AL;
 CONSULTANT;
   CONSULTING.ENGR = Tuskegee Institute;
   ADDRESS = Tuskegee;
   STATE = AL;
   ZIP = 36088;
```

APPENDIX G

VARIABLE-GRADE GRAVITY SEWER INSTALLATIONS

Table 2. Summary of Effluent Drain Projects Reviewed*

COMMUNITY	POP.	PHYSICAL FEATURES	NO CONNECTIONS	LENGTH						LENGTH/ CONNECTION	COMMENTS	DATE ON-LINE	DESIGN ENGINEERS
				TOTAL	3"	4"	6"	8"	PRES.				
Mt. Andrew, Alabama	100	Gently sloping	31	2,500 ft.	50% 2" / 3" 50%	—	—	—	—	81 ft.	•Inflective gradient with sections depressed below HGL •2-in. minimum diameter drains •No manholes or cleanouts •Some pressure inlets	July, 1975	Rural Housing Research Unit & Farmers Home Administration USDA
Westboro, Wisconsin	200	Gently sloping. Deep well drained soils	87	18,846 ft	—	77%	—	5%	18%	217 ft.	•Uniform gradient •Curvilinear alignment between manholes •Hybrid gravity/pressure system	September, 1977	University of Wisconsin & Carl C. Crane, Inc. Madison, Wisconsin
Badger, South Dakota	105	Flat to gently rolling. Poorly drained soils. High seasonal water table	53	6,616 ft.	—	77%	23%	—	—	125 ft.	•Uniform gradient	November, 1980	Doss Engineering Huron, South Dakota
Avery, Idaho	90	Narrow, steep sided mountain valley bottom. Moderately deep soils	55	6,690 ft	—	100%	—	—	—	122 ft.	•Uniform gradient •No horizontal control maintained during construction •Pipe gallery reserve storage in lift stations	September, 1981	V. David Welch Assoc., Inc. Coeur d' Alene, Idaho
Maplewood, Wisconsin	150	Flat. Very shallow creviced bedrock	61	5,800 ft	—	—	100%	—	—	95 ft.	•Uniform gradient •Emergency pump manholes below each lift station	November, 1981	Foth & Van Dyke & Assoc., Inc. Green Bay, Wisconsin
South Corning, New York	2000	Flat valley bottom. Steep side slopes. Poorly drained soils	606	42,525 ft	—	77%	23%	—	—	70 ft.	•Uniform gradient •"Sump manholes" isolate sections of network	July, 1983	Philip J. Clark, P.E. Corning, New York
New Castle, Virginia	190	Gently sloping. High seasonal water table. Bouldery soil	64	6,955 ft.	—	64%	—	36%	—	109 ft.	•Uniform gradient	May, 1982	Anderson & Assoc., Inc. Blacksburg, Virginia
Miranda, California	300	High, moderately to steeply sloping river terrace. Deep alluvial soils	100	9,617 ft.	9%	91%	—	—	—	96 ft.	•Uniform gradient	November, 1982	Winzler & Kelly Eureka, California
Gardiner, New York	500	Gently sloping	109	19,330 ft	—	71%	19%	10%	—	177 ft.	•Uniform gradient •Curvilinear alignment between manholes and cleanouts	December, 1982	Erickson & Silber Engineers Goshen, New York
Lafayette, Tennessee	1500	Top of plateau and steep side slopes	510	45,310 ft	—	47%	53%	—	—	89 ft.	•Uniform gradient •Curvilinear alignment between manholes and cleanouts •Aerated lift stations for odor control	September, 1983	John Coleman Hayes & Assoc. Nashville, Tennessee

REFERENCE: Otis, R.J., "Small Diameter Gravity Sewers: An Alternative Wastewater Collection System for Unsewered Communities", USEPA MERL, Cincinnati, Ohio, 1985.

APPENDIX H

MORRO GROUP LETTER REPORT RE DISPOSAL SITE 6

THE MORRO GROUP

January 9, 1985

Office of the Environmental Coordinator
County Government Center
San Luis Obispo, CA 93402

ATTN: Mr. Vincent Morici

SUBJECT: Geological and Geophysical Investigations of Proposed Disposal Site 6 and Conceptual Infiltration Model, CSA 9 Wastewater Treatment Facilities

Dear Mr. Morici:

The following report summarizes the results of geological and geophysical investigations of proposed wastewater disposal Site 6. The geological investigation of the site and adjacent areas to the south was undertaken concurrently with with the Phase 1 soils investigation conducted by Pacific Geoscience. The geophysical investigation was conducted as a part of Phase 2 to further refine the thickness of the wind-blown sand unit on the site. The primary objective of the geological and geophysical investigations has been to evaluate the feasibility of wastewater disposal and groundwater recharge at Site 6. The conceptual infiltration model has been developed for use in conjunction with the soils data in this evaluation.

RESULTS OF INVESTIGATIONS

Geological Characteristics of the Site

Site 6 is located near the southern edge of the sheet of wind-blown sand that extends upslope from the old sand dunes that underlie Baywood Park and the northerly portions of Los Osos. As a result, the surficial geologic unit at the site is almost everywhere composed of poorly consolidated, fine sand with relatively small amounts of silt and essentially no clay.

The wind-blown sand unit is underlain at varying depths by moderately consolidated sandstone, siltstone and claystone assigned herein to the Paso Robles Formation. The Paso Robles Formation also crops out as local patches in the extreme south portion of the site and as a more continuous band at the crest of the ridge immediately south of the site (Figure 1). Additional outcrops may be present within the mapped

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area. However, because of the biological sensitivity of the site and the dense cover of brush on most of its southerly portion, the investigation was limited to existing trails and open areas immediately adjacent to these trails. The Paso Robles Formation was also identified in borings 1, 2, and 5 by the presence of clayey siltstone and silty claystone. It may also be present in the other four borings drilled during the soils investigation. However, distinguishing Paso Robles sand from the wind-blown sand in samples recovered during drilling is difficult, and a reliable identification of Paso Robles Formation in the remaining four borings was not possible.

The Paso Robles Formation is underlain by the Miguelito Member of the Pismo Formation which consists primarily of relatively hard and resistent beds of siltstone and claystone. This unit crops out along the ridge to the south of the site, but it was not encountered in any of the borings drilled during the soils investigation.

Results of the Geophysical Investigation

Twenty-nine seismic refraction profiles were shot at and adjacent to the site to further delineate the thickness of the wind-blown sand unit. The first 5 profiles (4 at borings and one on the Paso Robles outcrop) were shot during Phase 1 to test the reliability of the method, and the remaining 24 profiles were shot as a part of Phase 2 investigations. These data indicate that the seismic velocity of the wind-blown sand unit is approximately 1100-1300 ft/sec. while that of the Paso Robles Formation is approximately 2000-3000 ft/sec. The change in velocity is "sharp", indicating a significant change in induration of the soil/rock units, and also that the change in velocity may be a more reliable indication of the top of the Paso Robles Formation than drilling samples.

The locations of the seismic profiles, the indicated thickness of the wind-blown sand unit, and contours on the thickness of this unit are shown on Figure 1. These data indicate that the wind-blown sand unit is 25-40 feet thick on the western third of the site, that it is 15-30 feet thick on the northeasterly third of the site, and that it is 0-15 feet thick on the southeasterly third of the site.

In addition to investigating the wind-blown sand thickness, the profile nearest the northeast corner of the site was extended to the maximum capability of the equipment available (300') to test for possible shallow groundwater in this area. A seismic velocity indicative of saturated sand (i.e., greater than 5,000 ft/sec.) was not observed, indicating that groundwater beneath this part of the site is at a depth greater than 117 feet below the surface. Groundwater beneath the northwesterly portion of the site is known to be below 150 feet from monitoring of well 24A1 located near the south end of Alexander Avenue.

CONCEPTUAL INFILTRATION MODEL

Geometry of the Geological Units

A cross section of a conceptual infiltration model based on the geological and geophysical investigations conducted on Site 6 is shown on Figure 2. The surface slope is shown at 12% toward the north which is about average for the site. The slope is steeper (about 14-16%) on the southwesterly third of the site, and more gentle (8-10%) near its easterly and northerly fringes. The thickness of the wind-blown sand unit is shown as being approximately 20 feet, and thickening downslope toward the north. This unit is thicker (30-40 feet) beneath the westerly and northerly portions of the site (Figure 1), and thinner beneath the southeasterly part of the site.

Bedding within the underlying Paso Robles Formation is presumed to be inclined toward the north at approximately 18% (10^0) based on the dip of this formation in an outcrop near the southeast corner of the site, and northerly dips in this general range mapped by Hall, Ernst, Prior and Wiese (1979) in Paso Robles outcrops along the south flank of the basin. The inclination of bedding exposed in the largest of the Paso Robles outcrops on the site is difficult to determine, but would also appear to be northwesterly at approximately $8-10^0$. Northerly dip at about 10^0 is consistent with correlation of the clay beds in this outcrop with a clay zone in Boring 2 at 40-50 feet and in the USGS South Broderson well at 346-370 feet. This inclination would facilitate infiltration of percolated wastewater from the wind-blown sand unit into the Paso Robles beds. However, it is probably not critical to the overall determination of the feasibility of disposal/recharge at the site provided at least some of the Paso Robles beds at the base of the wind-blown sand unit are permeable sand. Since the upper 207 feet of section encountered in the USGS well is primarily sand, this would appear to be a reasonable assumption.

Geometry of the Infiltration Basins

Because of the moderately steep slopes of Site 6, it is assumed that the infiltration basins would be constructed as elongate trenches oriented approximately parallel to the surface contours (i.e., near east-west) rather than as typical ponds with roughly equal dimensions. Also, subsurface conditions are such that dispersal of the infiltrating wastewater to the groundwater system will be facilitated by a basin configuration that is as long as is otherwise feasible in a direction parallel to the surface contours. For purposes of the conceptual model, a typical infiltration basin is assumed to be approximately 10 feet deep, have a bottom width of 60 feet, and be constructed approximately as shown on Figure 2.

Rates of Infiltration

Based on the Pacific Geoscience report of December 4, 1985, percolation rates in the wind-blown sand unit are summarized as follows:

Boring	Percolation Rate (minutes/inch) at a Depth of: 10 feet	25 feet
1	0.2	1.0
2	0.7	1.0
3	0.1	5.0
4	2.7	4.0
5	0.1	-
6	0.1	0.1
7	0.1	0.1

The one percolation test conducted in sand of the Paso Robles Formation (at 25 feet in Boring 5) yielded a rate of 13 minutes/inch, which is about one order of magintude slower than the 25-foot tests in the wind-blown sand unit.

Based on these test data, Pacific Geoscience (Shallenberger, per. comm.) has suggested that a rate of 3 minutes/inch (40 feet/day) would be reasonably conservative for the percolation rate parallel to bedding in fresh exposures of the the wind-blown sand unit. While testing of the Paso Robles sands was limited to one boring, it would appear reasonably conservative to assume for modeling purposes that the percolation rate for this formation is about 1/10th the rate for the wind-blown sand unit.

Rates of Infiltration as Applied to the Conceptual Model

There are two "critical windows" through which the infiltrating wastewater must pass given the configuration of the model as depicted on Figure 2. The first is the bottom of the percolation trench, annotated Q_1 on Figure 2, and the second is the downslope window, annotated Q_2, between the bottom of the trench and the top of the Paso Robles. If flow Q_1 is assumed to be vertical, then it would be primarily across bedding, and a reduction in the infiltration rate would have to be considered. Factors of 1/10th the horizontal rate have been suggested as being conservative for this assumption even though the wind- blown sand unit is relatively homogenous in comparison to most bedded sand soils. On the other hand, the bottom of the trench will intersect the sheet sand at an angle of 12%, and an upper limit on the reduction in infiltration would appear to be a factor of 1/8th of the tested rate (5 feet/day) for fresh exposures in the bottom of the trench.

An alternative approach would be to consider rates of infiltration that can be derived from permeabilities for wind-blown sand available in the literature. Lambe and Whitman (1969, p. 287) give permeabilities of dune sand as being in the range of 0.1-0.3 cm/sec. If the lower end of this rate is assumed, i.e. 0.1 cm/sec. or 283 ft/day, and the slope is 12%, then a rate of infiltration, laterally and parallel to bedding in the sheet sand and also the surface slope, of 34 ft/day can be interpreted as being possibly applicable to the computation of infiltration rates. This value is very close to the 40 ft/day interpreted as a conservative rate from the percolation tests. This, in turn, suggests that a rate in this approximate range is applicable whether it be vertical and across bedding as for the bottom of the trench, or whether it be on a slope of 12% and parallel to bedding which can be applied to the downslope window or to the bottom of the trench as an upper limit for fresh exposures.

Based on these values, and assuming a trench width of 60 feet and average wet-weather loadings of 2.0 mgd (6.14 ac-ft/day), a trench length of only 111 feet would be required <u>if the trench bottom would maintain its initial permeability throughout its period of use</u>. More realistically, a trench length of about 1,000 is considered minimal which would provide a safety factor for deterioration of trench-bottom permeability of approximately 10. Also, it will be necessary to distribute infiltration along a length of approximately 1,000 feet if flow through the critical downslope window is to be provided for. Based on the percolation/permeability factors developed above and assuming a trench length of 1,000 feet, the height of the water column through window Q_2 of Figure 2 would be approximately 6.7 feet. This can be provided for in the design of the infiltration trenches if the wind-blown sand unit is at least 15-20 feet thick. If the trenches were to be excavated on a thinner wind-blown sand section, then downslope surfacing of wastewater may occur, and the stability of the downslope berm of the infiltration trenches would be highly suspect.

These preliminary considerations suggest that trench-bottom permeability and deterioration with use is not the critical design factor, but rather that maintenance of an adequate downslope window within the wind-blown sand unit is the critical design constraint. Available data suggest that 15 feet of wind-blown sand would be minimal for a 1,000-foot trench length, and that at least 20-25 feet of wind-blown sand would preferable.

<u>Downslope Infiltration to the Groundwater System</u>

In the discussion above, it is assumed that the wastewater would flow downslope within the wind-blown sand section for an indefinite distance. In actual application, the percolated wastewater will gradually infiltrate downward into the Paso Robles Formation as, at the northerly edge of the site, the groundwater table is approximately 100 feet below the top of this unit. Assuming infiltration into the Paso Robles at a rate 1/10th that of the wind-blown sand unit and a functional trench bottom of 60 feet, all of the wastewater percolated into the wind-blown sand unit would infiltrate into the upper Paso Robles Formation within a distance of approximately 600 feet downslope from the percolation trenches. Even if these estimates were to be off by a factor of 10 or more, there should still be no surfacing of percolated wastewater in the developed area downslope.

If you have any questions on the results of these investigations, please call me at 805/528-2187.

Sincerley,

Donald O. Asquith

Donald O. Asquith
Engineering Geologist, EG-913
Registered Geophysicist, GP-86

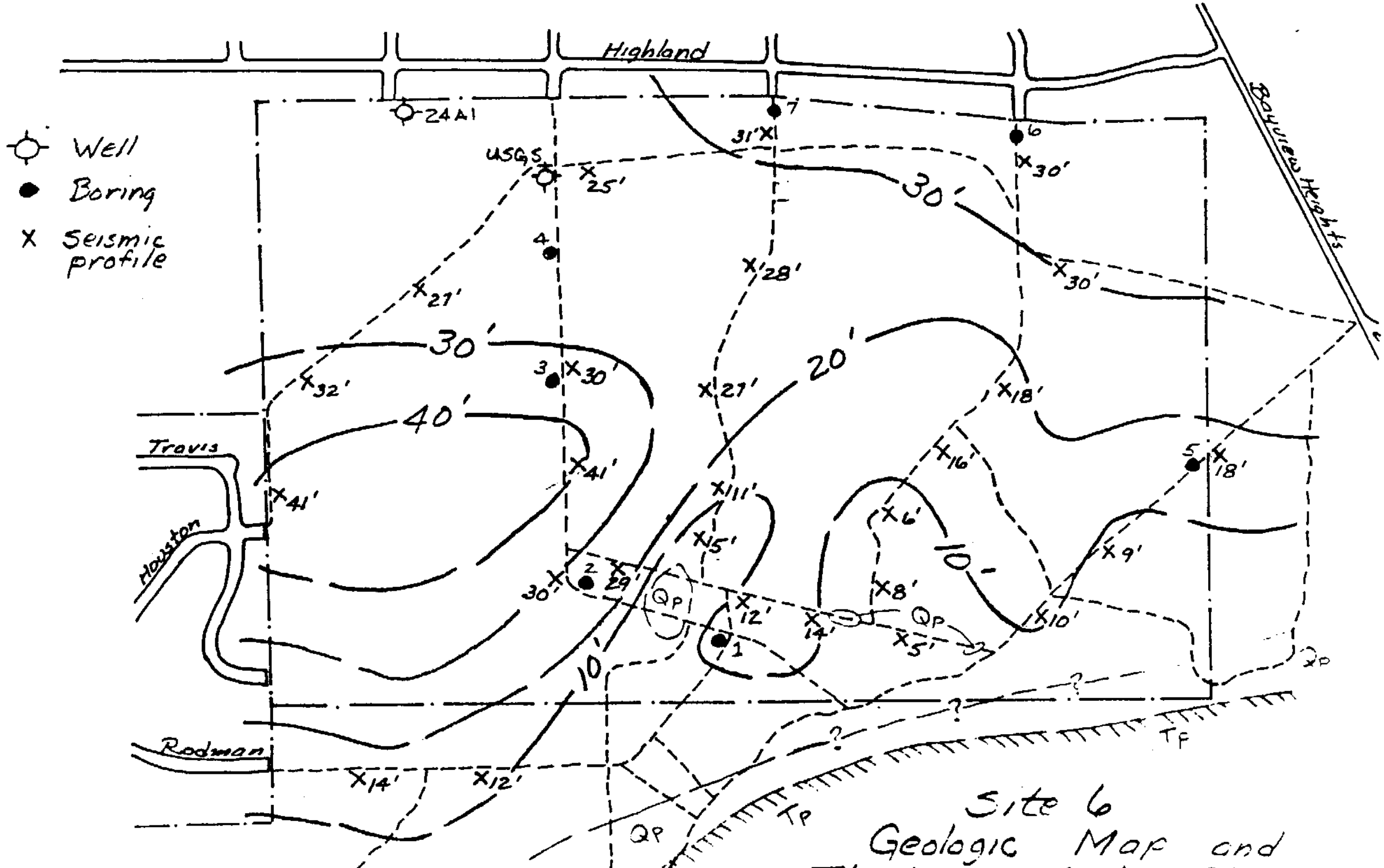
cc: Pacific Geoscience

REFERENCES

Hall, C. A., W. G. Ernst, S. W. Prior, and J. W. Wiese, 1979, Geologic map of the San Luis Obispo-San Simeon region, California: U. S. Geological Survey Map I-1097.

Lambe, T. W. and R. V. Whitman, 1969, Soil Mechanics: John Wiley & Sons, Inc.

Figure 1



Site 6
Geologic Map and
Thickness of Wind-Blown Sand
1" ≈ 500'

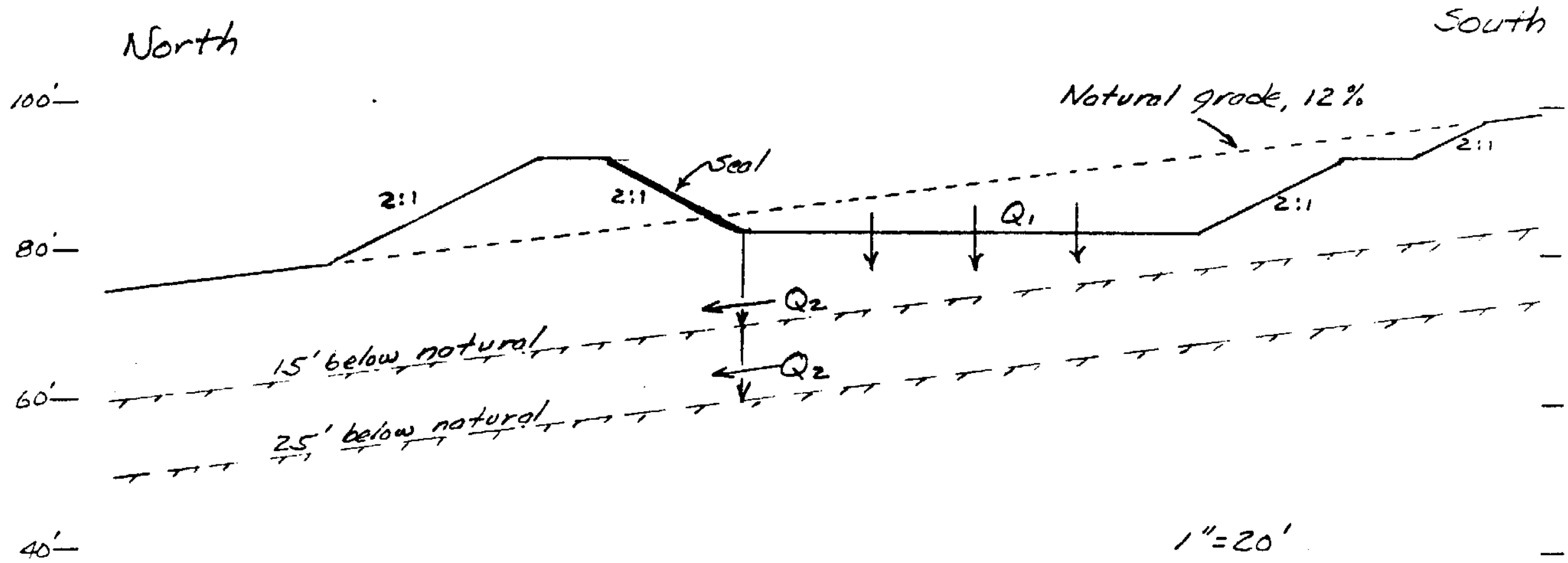


Figure 2
Conceptual Infiltration Model

APPENDIX I

SLO COUNTY MEMORANDUM RE BLACK LAKE SEPTIC TANK EFFLUENT

RECEIVED
APR 9 1986
ENGINEERING-SCIENCE, INC.
MONTEREY, CALIFORNIA

July 29, 1981

MEMORANDUM

TO: Hal Wilkinson

FROM: Percy Garcia

SUBJECT: Treatment of Black Lakes Septic Tank Effluent With Sulfur and Lime

Following are the results of the recent treatment with sulfur and lime of the Black Lakes septic tank effluent:

Constituents	Source	6-17-81 State Eff.	6-16-81 SO_3 + Lime
Bacte. Analysis		$\geq$ 240 $\geq$ 240	#1 Coliform $\geq$ 240 Fecal 15
TDS	693	1,149	1,881
Cl	61.7	169	81.9
Na	79	144	174
SO_4	256	215	776
NO_3		4.43	0.35
NH_3-N		2.71	#1 = 31.6 #2 = 27.9 #3 = 5.7
Total Kjeldahl	0.6	43.5	38.2
C.O.D.		138	268
B.O.D.		170	105
Sus. Solids		10	8

The samples were collected by Doug Jones.

I.

1. The TDS increased 63%.
2. The Na increased 21%.
3. The SO_4 increased 261%.
4. The NO_3 reduction = 92%.
5. The NH_3-N increased 110%.
6. The Total Kjeldahl reduction = 12%.
7. The C.O.D. increased 94%.
8. The B.O.D. reduction = 38%.
9. The Suspended Solids reduction = 20%.

II.

By adding sulfur and lime we observed the increases on:

TDS	=	63%
Na	=	21%
SO_4	=	261%
NH_3-N	=	110%
C.O.D.	=	94%

Reductions on:

NO_3	=	92%
Total Kjeldahl	=	12%
B.O.D.	=	38%
Suspended Solids	=	20%

The addition of the two elements of sulfur and lime, while causing a reduction in some elements, had an overall detrimental effect of increasing other key elements. What appeared to be a reduction, for instance, in nitrates, was actually a chemical change of nitrates to ammonia. The bacterial count was quite high.

The sampling, done by Doug Jones, was not taken in stages. Because only the effluent was actually sampled, it is impossible to determine where the problem with the treatment actually appeared.

PG/dk